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INCLUDING PRESTRESSED CONCRETE

JANUARY 1958 **VOL. LIII. NO. 1**

CONCRETE SERIES

FIFTY-THIRD YEAR OF PUBLICATION

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Volume LIII, No. 1. LONDON, JANUARY, 1958.

EDITORIAL NOTES

The Cost of Designs.

IN most branches of engineering, the detailed design of a machine or structure involves some repetitive calculation, in which the form remains unchanged but certain of the assumed values are altered, such as, for example, the stiffnesses of the members of a rigid frame. These calculations, which are not difficult in themselves, are frequently tedious to the computer and can account for a large part of the cost of designing reinforced concrete structures. Consequently many engineers are content to check their designs to make sure only that their initial assumptions do not result in members that are overstressed or are too flexible, or for which the load-factor is too low, and except in the case of important structures designers are seldom able to afford the time and cost of preparing and comparing the efficiency of several designs. The efficiency and economy of the resulting structure are thus determined to some extent by the intuition and skill of the designer.

There is no doubt that this sytem is satisfactory in the case of most structures, for which the calculations are generally conventional and bear little relation to the stresses which are likely to occur in the structures. The ability to make an accurate forecast of the sizes of members necessary to resist the forces and moments applied to them can be developed to a high degree, and a good guess by a designer with such ability is probably preferable to an intricate calculation by a designer without it ; it is easy for a designer with little engineering sense to spend ten pounds' worth of time to save five pounds' worth of steel. Nevertheless, the need for such calculations occurs with sufficient frequency to make the use of labour-saving methods attractive, although the number of such devices which have been developed or adapted is probably out of proportion to the importance of the problem.

Probably the most widely-used group of these methods is that which involves the use of a graphical procedure. The simplest form is the Cartesian graph, by means of which an equation with a single variable can be solved. Such solutions have been sought from early times ; one of the earliest graphical methods was devised for the solution of problems relating to spherical triangles by Hipparchus in the second century B.C. The usual form of graph is, however, restricted in its application, and a greater generality was brought to graphical solutions

with the introduction of nomograms. A nomogram is a diagrammatic representation of a formula in which the variables are represented by a set of graduated lines from which the solution of the formula may be obtained by an index line.

The introduction of nomograms to engineering is largely due to a Frenchman, Phillip Morice d'Ocagne, who was professor of topometry and applied geometry at the École des Ponts et Chaussées at the end of the nineteenth century, and who for his work on the subject was awarded the Prix Poncelet in 1902. It may indeed be said that nomography began with the publication in the year 1891 of d'Ocagne's book "Les calculs usuels effectuees au moyen des abaques." Since that time nomograms have been used extensively in applied science and engineering, but their application to the problems of structural engineers has not been dealt with comprehensively. In a recently-published work,* however, nomograms are given for use in the calculation of thrusts, bending moments, and shearing forces in statically-indeterminate frames; these nomograms relate to the design of such frames with eight different types of loading, namely concentrated, uniformly-distributed, and triangular loads on the beam; concentrated, uniformly-distributed, and triangular loads on either or both of the columns; and concentrated loads acting on the beam or on a column. Such problems are now of everyday occurrence in reinforced concrete, and with the increased application of welding, as well as of such methods as the use of high-tensile pre-loading bolts, are likely to become as common in structural steel.

It is probable that the greatest value of such methods lies in their use for the quick checking of designs. It is most desirable that a design should be checked by someone other than the man who prepared it, but because of the time required there may often be a temptation not to do so. Now that rapid methods of checking are available this excuse is no longer valid and some of the costly delays and revisions which occur when errors are found at a late stage could be eliminated by careful checking. Arithmetical computations can never be entirely eliminated, but they can and should be reduced as much as possible. They represent the least important and least interesting part of a designer's work, and it is becoming increasingly common for such work to be done by specially-trained assistants, skilled in computational methods and working under the direction of the designer, thereby leaving the designer free to concentrate on the more important aspects of design. This is much more desirable than the equally common practice of allowing junior members of an office to use graphical or other methods which they may not understand to produce results which they are frequently unable to interpret. The number and variety of mathematical methods which are now available is so large that their application is increasingly becoming possible only to those trained in their use.

The problems of calculation which are of greatest importance to a designer are not those which require the solution of a small number of equations but those which require rapid and accurate approximations, and these in general have attracted much less attention than the former. The time saved by using a graphical method of solution of equations is usually to be measured in minutes, but a rapid and reasonably accurate method of assessing, for example, the effects of the wind on a multiple-story structure, or the distribution of stresses in a shell roof, would often save days, and sometimes weeks, of calculation.

* "Nomograms for the Analysis of Frames." By J. Rygol. Concrete Publications, Ltd. 1957. Price 18s.

Industrial Structures in Poland.

By PROFESSOR W. OLSZAK and S. KAJFASZ.

WARSAW

THE design of the reinforced and prestressed concrete structures built in Poland in recent years has been influenced by the scarcity of structural materials, and this gave a strong stimulus to the use of methods that would be economical in the use of materials. In the case of industrial structures it was also desirable that as many standard designs as possible should be produced and used for many structures. For these reasons the Bureau for Studies of Standard Building Designs (directed by Mr. J. Kawecki, Mr. K. Husarski, and Mr. B. Kowalski) was established, and its recommendations are tested experimentally by the Institute for Building Techniques. In each case loading tests to failure are made on full-size models. It is thus possible to ascertain the deflections and deformations and the safety factor of a prototype of every structure, to find any weak points, and to gain information on the causes of failure, so that the designs can be improved and the best selected before it is recommended as a standard. The results are published by the Bureau for Studies of Standard Buildings Designs.

In this article prestressed concrete structures, thin shell roofs constructed on moving scaffolding, and completely precast roofs with lattice trusses are discussed, together with special industrial structures in which the efficiency of the construction is the result of the use and efficient assembly of precast members.

Shell Roofs.

The three-dimensional characteristics of the loading of a shell roof and its construction in separate precast elements appear to be contradictory, and the establishment of continuity by joining the elements to form a monolithic structure is a laborious and costly process that more than counteracts the advantages of prefabrication. When a standard element of a shell roof can be repeated many times, it is therefore better to abandon the idea of prefabrication and to use travelling scaffolding that enables concreting, steam curing, and other processes to be combined in one operation. For example, the building (*Fig.* 1) for the repair of railway rolling stock designed by Messrs. W. Zalewski, J. Draguła, and Z. Walczyna is a conoidal shell structure 250 ft. by 440 ft. supported by columns on a grid of 23 ft. by 83 ft. The columns are rigidly fixed in the foundations. The edge stiffening arch ribs of adjoining conoids are interconnected by steel or reinforced concrete tie-members to form windows. The shell is 3 in. thick. The quantity of steel used was 1·54 lb. per square foot of the area covered by the

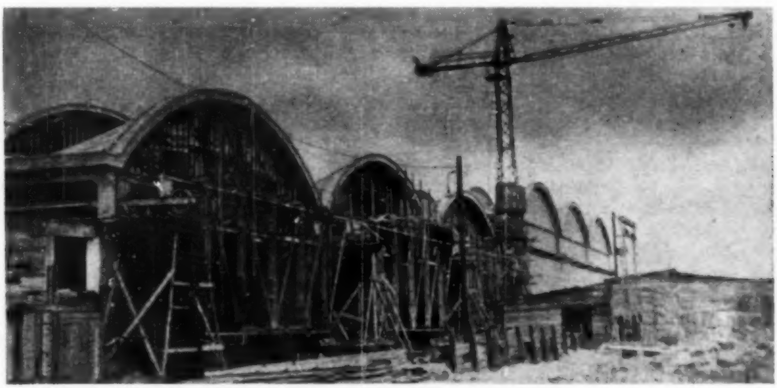


Fig. 1.—A Conoidal Shell Roof.

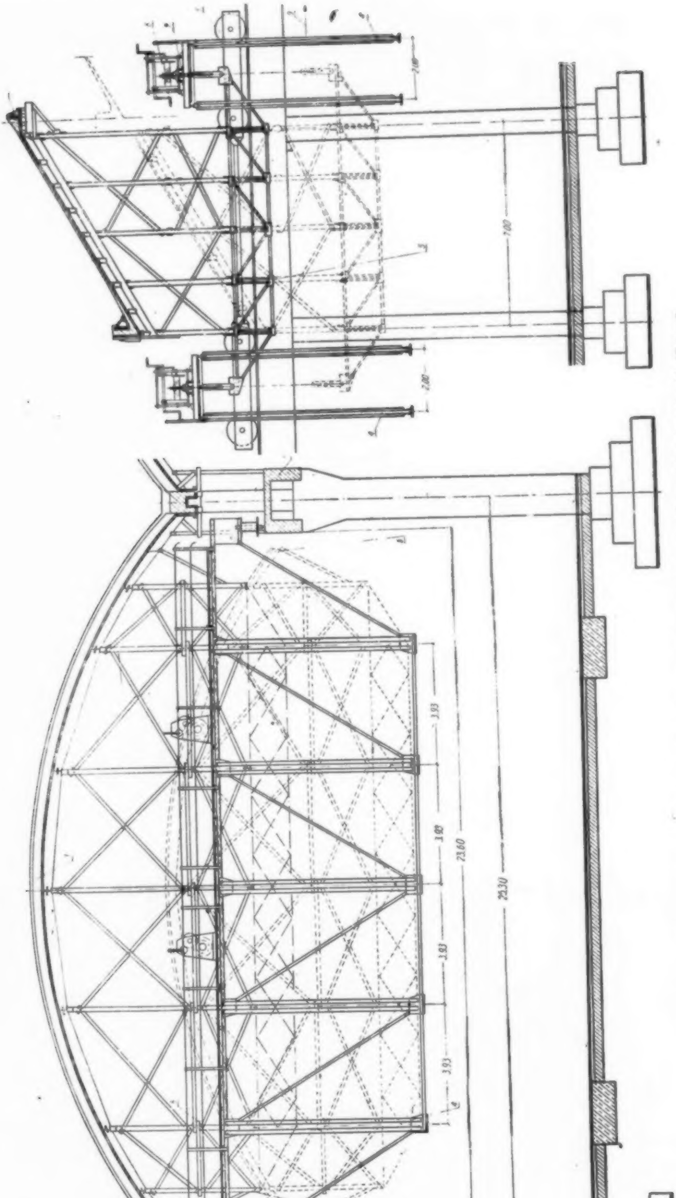


Fig. 2.—Travelling Falsework for constructing a Conoidal Shell Roof.

shell and 4·2 lb. per square foot in the edge-beams. The quantity of concrete was 0·256 cu. ft. per square foot in the shell and 0·73 cu. ft. per square foot in the edge-beams.

The foundations, columns, and crane girders were constructed by the usual methods before the shell was concreted, and the shell was constructed by means of a travelling falsework moving on the crane girders. The shuttering was supported on four steel lattice girders (*Fig.* 2). The vertical members of the lattices were strengthened to act as guides for a vertically-movable platform on which was supported a wooden scaffolding and the centering formed to the shape of the under surface of the conoid. The vertical movement was obtained by means of four ordinary hand-operated cable-hoists with a lifting height of 15 ft. 6 in. The scaffolding, which was supported on four trucks moving on single rails, was moved laterally by means of pulley blocks.

The steam-curing installation is provided on the travelling scaffold. The construction of each bay was completed in six to eight days, including the lowering of the platform with the scaffolding, the horizontal movement to the new position, raising the platform, adjusting the centering, concreting, and steam curing. The scaffolding comprises about 60 tons of steel and about 20 tons of wood.

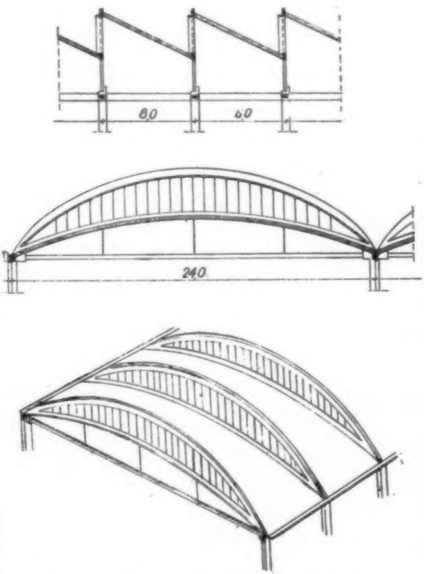


Fig. 3.—Conoidal Shell with Tie.

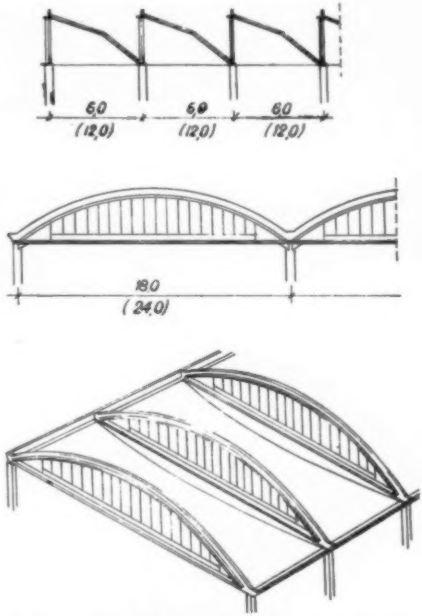


Fig. 4.—Conoidal Shell with "Broken" Surface.

After the first structure was built the conoid was redesigned with a column spacing of 85 ft. by 20 ft., and the thickness of the shell was reduced to 2·4 in. The dimensions of the beams were also reduced, and additional ties were introduced in the plane of the skylight windows (*Fig.* 3). These alterations reduced the quantity of concrete to 0·46 cu. ft. per square foot and of steel to 3·2 lb. per square foot. In one of the designs the steel ties were replaced by prestressed concrete ties, which were more rigid and further reduced the consumption of steel.

In the next design the shallow arch was replaced by a horizontal beam and the generant lines were "broken" by the introduction of an intermediate arc (*Fig.* 4). This type of structure was designed for column spacings of 60 ft. by 20 ft., 60 ft. by 40 ft., 80 ft. by 20 ft., and 80 ft. by 40 ft.; in the case of the spacing of 80 ft. by 40 ft. the quantity of concrete

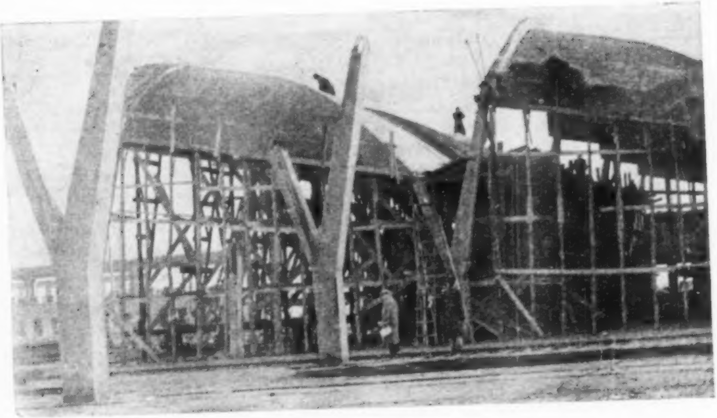


Fig. 5.—Semi-cloister Vaults.

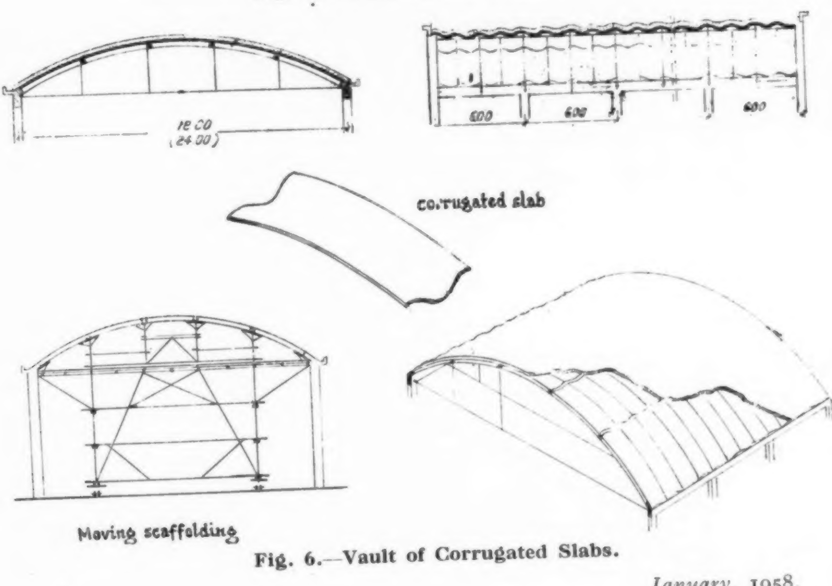


Fig. 6.—Vault of Corrugated Slabs.

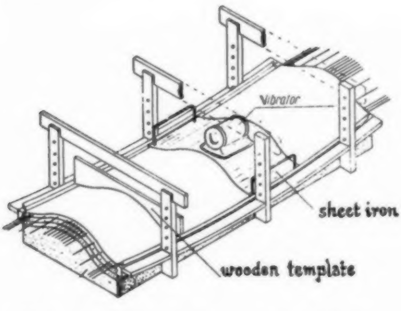


Fig. 7.—Casting Corrugated Slabs.

is 0·36 cu. ft. per square foot and of steel 3·05 lb. per square foot.

In the development of conoidal shell structures, the design (*Fig.* 5) of Mr. W. Zalewski should be noted. It is in the form of half of a cloister vault and can be used (as well as the conoidal shell shown in *Fig.* 4) for structures on a grid of square or rectangular (1 : 2 side ratio) column spacing. This design can be considered as intermediate between a conoidal shell and a "shed" shell roof of the Zeiss-Dywidag type. Its advantage is that it preserves the advantages of a "shed" type of roof without the use of rigid longitudinal upper edge-beams.

Corrugated Slab Roofs.

The rigidity of cylindrical corrugated vaults is well known, and vaults of this type have often been used, for example by M. Freyssinet in the hangars at Orly. Corrugated vaults were used recently for a spinning mill at Łódź, the engineers for which were Messrs. S. Klimek and W. Meuš. It was constructed of precast slabs measuring about 3 ft. 3 in. by 9 ft. by 2 in. thick and weighing 770 lb. The span of the roof (*Fig.* 6) was 70 ft. The slabs were made in stacks (*Fig.* 7); the lower slabs formed the base of the slab above them and were kept apart by layers of clay about ⅛ in. thick. The shape of the arch made it possible to use one shape of slab only, and they were joined by welding the ends of steel bars protruding from the shorter sides of the slabs and concreting the gaps between them. The longer sides were jointed with mortar without steel reinforcement. The joints are at the tops of the corrugations. Thermal insulation is obtained by wood-wool slabs over the concrete.

The slabs were erected by means of a movable wooden scaffolding 30 ft. wide, and a bay 30 ft. by 70 ft. was completed in five days. The slabs were covered with two layers of tarred paper. The quantities of materials required were 0·23 cu. ft. of concrete and 1·05 lb. of steel per square foot of the horizontal surface for the slabs and 0·72 lb. of steel per square foot in the ties. When a specimen of the vault measuring 20 ft. by 70 ft. was loaded to failure the factor of safety was 2·5.

Reinforced Lattice Trusses.

The truss shown in *Fig.* 8, used for the roof of a motor-car factory at Zeran, designed by Mr. S. Klimek, is a development of the so-called "Italian" girder, rectilinear girder elements being replaced by a parabolic arch structure. The axial forces in the bars of the girder are computed assuming a hinged lattice loaded by

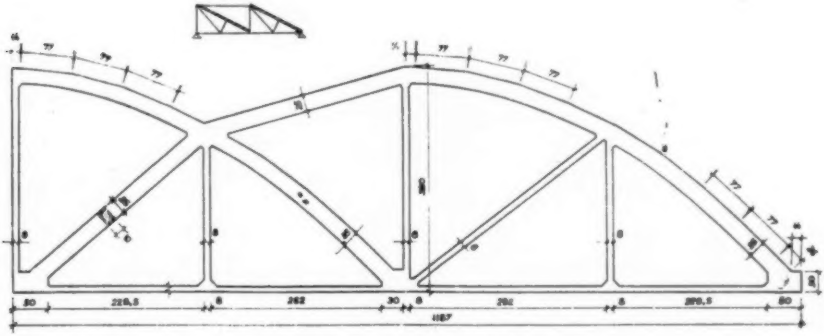


Fig. 8.—A Lattice Girder.

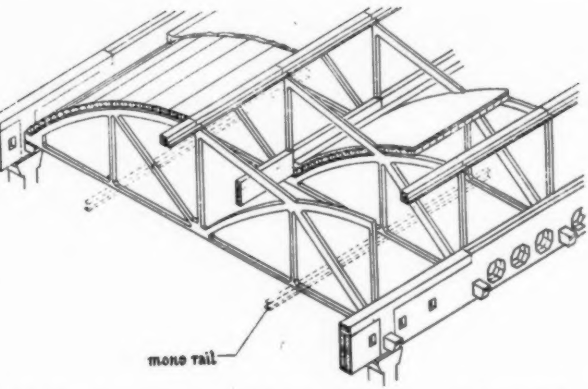


Fig. 9.—Arrangement of Lattice Girders and Covering.

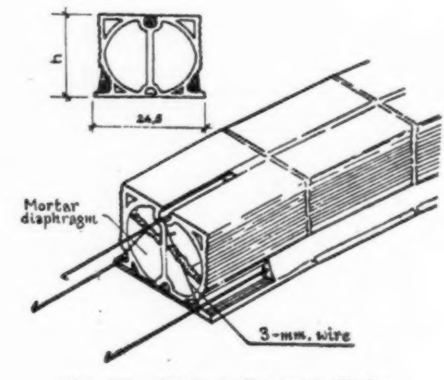


Fig. 10.—Hollow Ceramic Slabs.

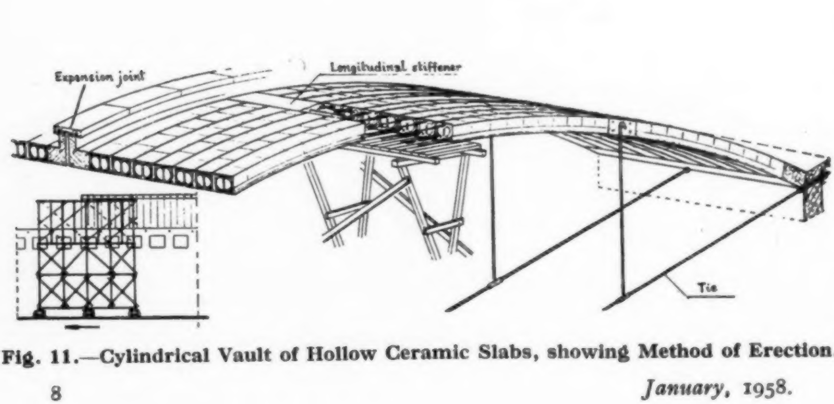


Fig. 11.—Cylindrical Vault of Hollow Ceramic Slabs, showing Method of Erection.

the wind, dead weight, snow, and a 2-tons hoist weighing 1100 lb. installed on a single rail suspended on hangers. The bending moments in the top boom due to its own weight and that of the roof covering and snow are computed as for a continuous beam of four or two spans. Instead of the top boom being composed of two straight members and a curved member, the whole flange can be made in the form of an arch; this gives a more pleasing appearance but requires more steel.

The girders were cast in stacks using sliding shuttering composed of 6-in. by ¾-in. boards stiffened with 4½-in. by 2½-in. battens. They were erected 10 ft. apart (*Fig.* 9) by means of a 3-tons crane. The thickness of a girder is 6 in. and the weight 2·6 tons. The quantities of materials are 0·26 cu. ft. of concrete and 2·85 lb. of steel per square foot. The girders are covered with reinforced hollow ceramic slabs made of DS-hourdis (*Fig.* 10); their weight is 51 lb. per square foot; this covering also stiffens the girders. Hollow slabs of this material have also been used by Mr. R. Dowgird as the structural elements for vaults (with ties) of 40 ft. span (*Fig.* 11). Roofs of this type for industrial buildings were very popular in the years 1949–1951, but they are not so popular now due to the difficulty of obtaining hourdis (clay) of suitable quality.

Prestressed Lattice Trusses.

Because prestressed roof trusses made with solid members were not economical, the problem was investigated by a team of engineers (Messrs. Z. Zieliński, St. Kuś, and A. Włodarz), directed by Mr. Wacław Zalewski, the designer of many original and outstanding structures. First, lattice trusses were designed composed of two booms, the upper boom being in compression and the lower boom in tension and prestressed. The booms are connected by vertical hinged ties. The distance between the centroid axes of the booms is proportional to the bending moments due to the load, and the top boom is curved so that its axis coincides with the line of pressure. Hence, for a uniform load the shape of the truss is parabolic; for concentrated loads it is polygonal. The trusses may be of different

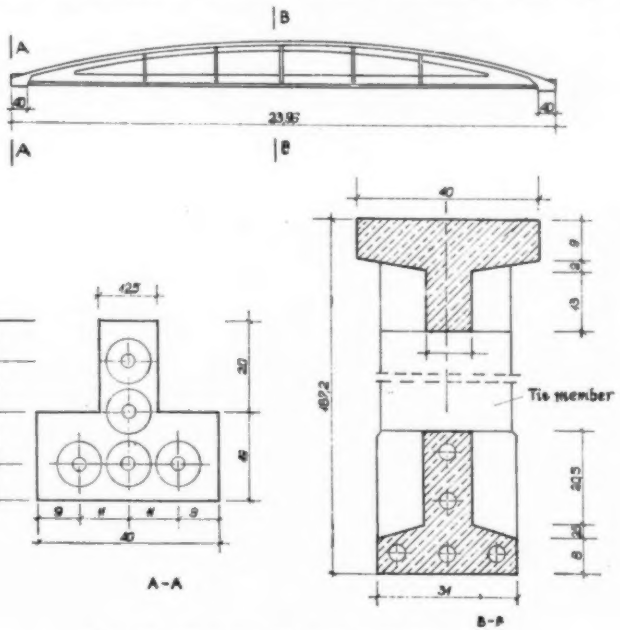


Fig. 12.—Truss with a Curved Top Boom.

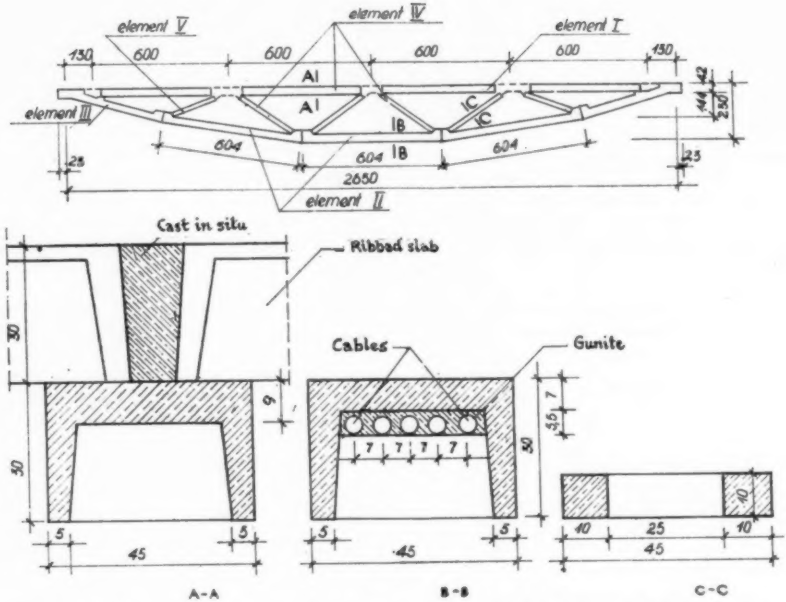


Fig. 13.—Truss with a Straight Upper Boom and a Curved Lower Boom.

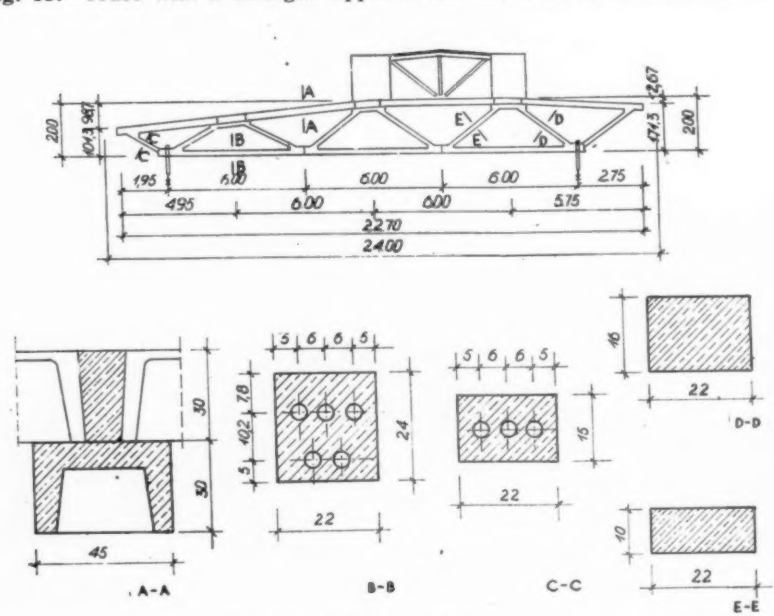


Fig. 14.—Self-stabilizing Truss.

shapes. Generally the upper boom is curved and the lower straight (*Fig.* 12); in some cases, for example for a flat roof or to increase the stability of the truss, the upper boom is straight and the lower curved (*Fig.* 13) or both are curved (*Fig.* 14). It must be borne in mind that as the girder does not have a solid web the area of concrete that will be subjected to prestressing is considerably reduced.

The analysis of the girder consists in determining the couple acting on the top and bottom booms at every cross section, and the checking of the stress in the booms considered as axially-compressed or tensioned members. The cross-sectional area of the prestressing steel is calculated by the ultimate-load method, a factor of safety of 2 being used. The stresses in both booms due to the dead and live loads are then checked. In determining the factor of safety against cracking (minimum 1·2) it is assumed that the first cracks appear when the tensile stress in the concrete is 1·7 times the axial tensile strength of the concrete; this means, for example, for "400" concrete (that is concrete with a compressive strength of 5700 lb. per square inch at 28 days) $1{\cdot}7 \times 384 = 654$ lb. per square inch. To allow for stresses during erection the factor of safety is increased to 1·4. Besides the normal live load the trusses are designed to carry an electric hoist (total load $3\frac{1}{2}$ tons) suspended from any point.

The cross-sectional area of the prestressed boom depends upon the permissible compressive stress in the concrete. The area of the compressive boom must allow for stresses occurring during the erection of the trusses and covering slabs. The U-shaped space formed between adjacent slabs is filled with concrete (*Fig.* 13), in which is embedded projecting reinforcement from the top flange. Thus the depth of the girder is increased as well as the area of the compressed part of the cross section, and the whole of the covering and the girders act as a monolithic structure.

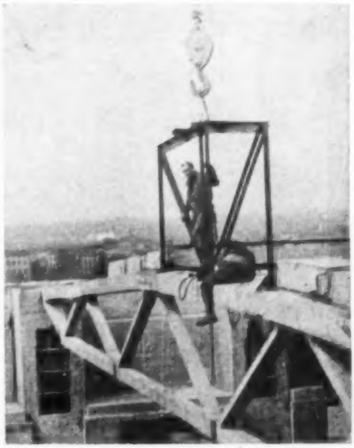


Fig. 15.—Erecting Truss and Roof Covering.

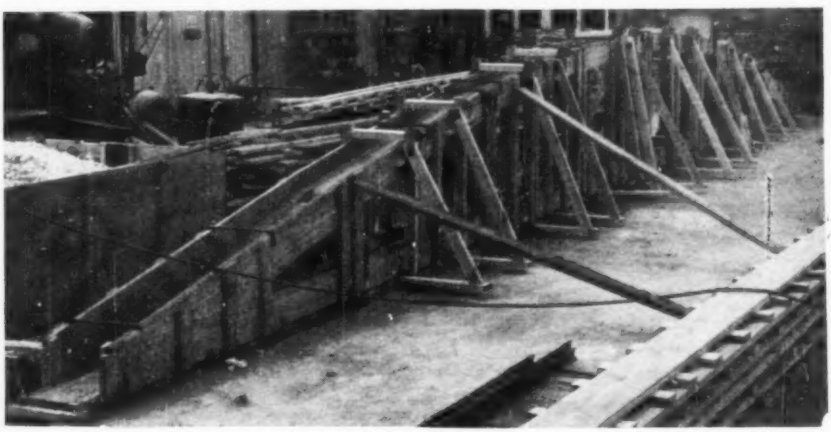


Fig. 16.—Mould for a Lattice Girder.

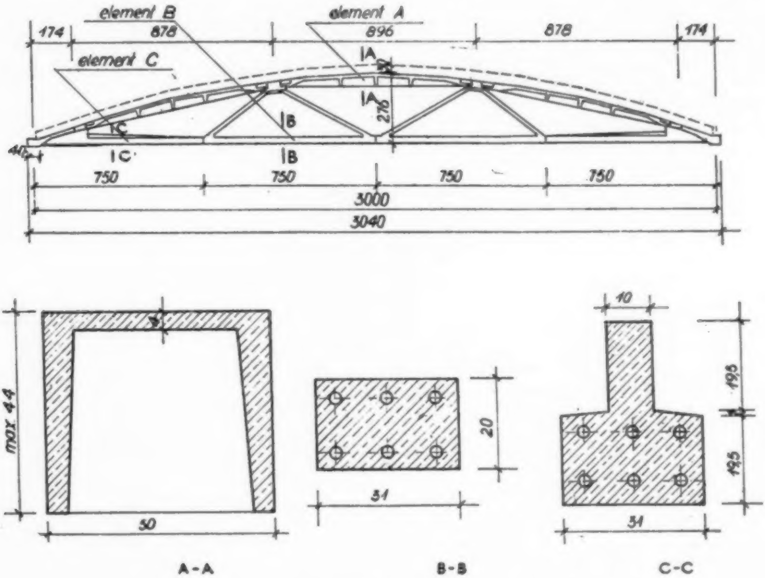


Fig. 17.—Truss divided into Triangular Sections.

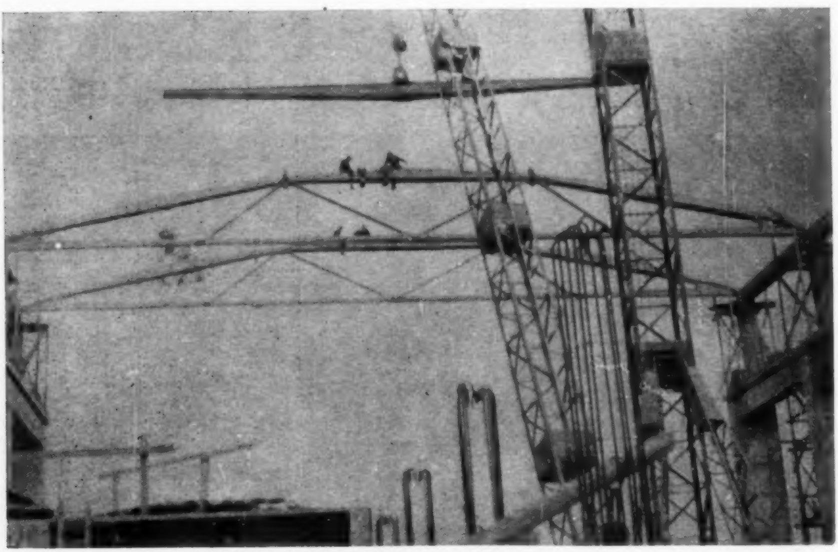


Fig. 18.—Erection of Lattice Trusses.

Vertical ties between the top and bottom members are sufficient to resist forces and moments due to a uniformly-distributed load. They do not, however, prevent additional bending moments on the booms due to other types of loads. For example, it may be necessary to provide temporary diagonal members to resist the forces that occur during erection ; in order to reduce these forces it is recommended to suspend the truss from as many points as possible.

The first trusses were made vertically in one piece in the mould shown in *Fig.* 16. With increasing spans and dimensions the trusses were made in parts that were joined by prestressing (*Fig.* 19). For trusses 100 ft. long, diagonal members were introduced dividing them into triangles and linear segments of the upper boom (*Fig.* 17). Each element could be made in the horizontal position on vibrating tables. In order to increase the rigidity of the elements of the truss,

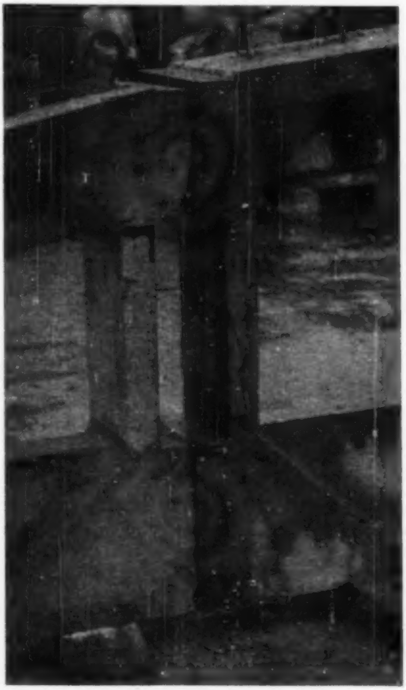


Fig. 19.—Detail of the Joint between the Parts of the Truss.

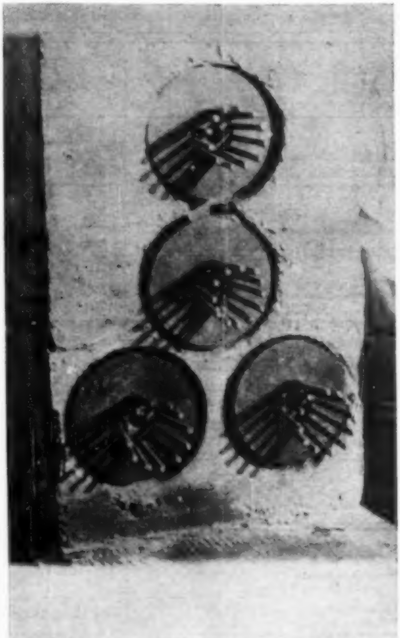


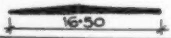
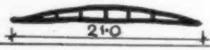
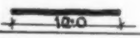
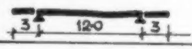
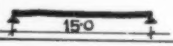
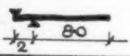
Fig. 20.—Anchors for Cables.

channel and tee-sections were used (*Figs.* 13, 14 and 17). *Fig.* 18 shows the erection of these trusses.

The joints of the compressed boom are filled with concrete cast in place and the segments of the tensioned boom are joined with butt straps. The lower boom is prestressed by six cables of the Freyssinet type. The anchors are steel blocks (*Fig.* 20) or reinforced concrete blocks. Trusses with a lower boom of parabolic shape have a channel-shape cross section with the open side downwards (*Fig.* 13), the prestressing cables being outside the concrete. This reduces the friction and facilitates the control of the tensioning of the cables. After prestressing the cables are protected by gunite.

All the types of trusses were tested at the Strzybnica factory (director Mr. A. Masłowski) under the control of the Institute for Building Techniques. Two trusses were laid side by side with a temporary platform and wooden boards (*Fig.* 21) and loaded with sand. The results are given in *Table* I. For

TABLE I.—RESULTS OF TESTS ON TRUSSES AND GIRDERS.

Roof trusses	Description of construction	Coefficient of safety against cracking real	Coefficient of safety against cracking theoret.	Coefficient of safety against failure real	Coefficient of safety against failure theoret.	Failure caused by
	I-shape, post-tensioned wire	1·47	1·49	2·63	2·54	concrete crushing
	Tied arch lattice truss (posttensioned)	1·59	1·49	2·37	2·34	—"—
	—"—	1·13	1·25	1·90	2·05	—"—
	—"—	1·55	1·45	1·90	2·00	—"—
	—"—	1·09	1·19	2·11	1·94	—"—
	—"—	1·06	1·16	2·25	2·20	—"—
	Prestressed I-beam	1·68	1·41	2·18	2·03	fracture of wires
	Prestressed I beam with r.c. cantilevers	1·65	1·42	2·37	2·05	—"—
	Prestressed I beam	1·57	1·43	2·30	2·33	—"—
	Prestressed electrical poles	1·42	1·34	2·08	2·09	—"—

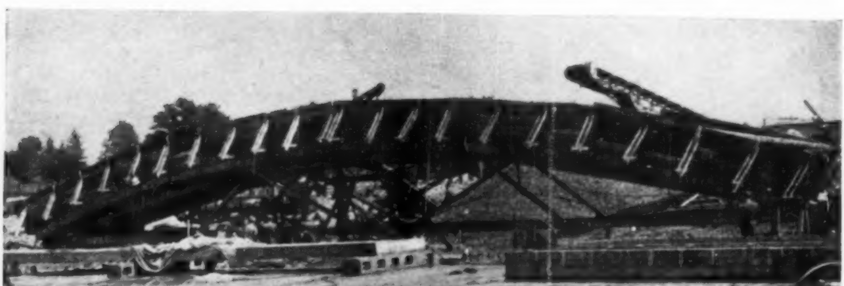


Fig. 21.—Method of Testing a Lattice Truss.

comparison, the results of tests of beams with pre-tensioned wires are also given.

The differences between the mechanism of failure of trusses with post-tensioned steel (large deflections causing crushing of the concrete due to the elongation of the steel) and trusses with pre-tensioned steel (failure due to rupture of the wire) is apparent, in spite of the fact that calculation of the ultimate strength indicated that the wire would break in all cases. Important factors, not yet considered in calculations, are the quality of bond between the wires and the concrete, which is much better with pre-tensioned wires, and the elongation of the wires, which is less for wires of smaller diameters.

The roof is covered with slabs measuring 5 ft. by 20 ft. (*Fig.* 22) with ribs 1 ft. deep and a slab 1 in. thick ; alternatively a grid of the same dimensions covered with slabs of foamed concrete 6 in. thick is used (*Fig.* 23), supported on frames (*Fig.* 24).

Lattice trusses are much used in industrial buildings, more than 700 having so far been made.

Prefabricated Structures.

The power station at Łódź (designed by Mr. W. Drużdżel and Mr. Z. Dytkowski ; Messrs. L. Suwalski and W. Zalewski, consulting engineers) makes use of large precast members. It has three bays 79 ft., 46 ft., and 89 ft. wide and is 660 ft. long (*Fig.* 25). The capacity of the building is more than 12,000,000 cu. ft. The heaviest precast elements are hollow columns 102 ft. high and 6 ft. 3 in. by 5 ft. 7 in. in cross section ; the walls are 6 in. and 8 in. thick and the columns weigh 96 tons (*Fig.* 30). The space in the column is used to conduct the air from above the boiler to the fire-box, thus saving 350 tons of sheet steel that would otherwise have been used for air-ducts.

The columns were cast horizontally of "200" concrete (that is concrete with a compressive strength of 2800 lb. per square inch at 28 days), in wooden moulds

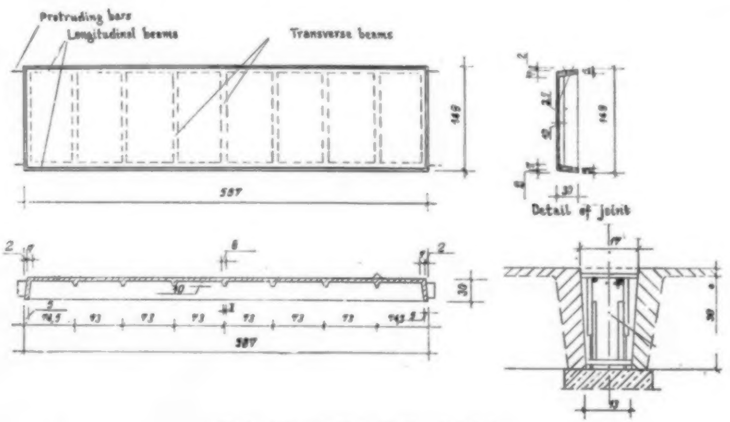


Fig. 22.—Ribbed Roof Slab.

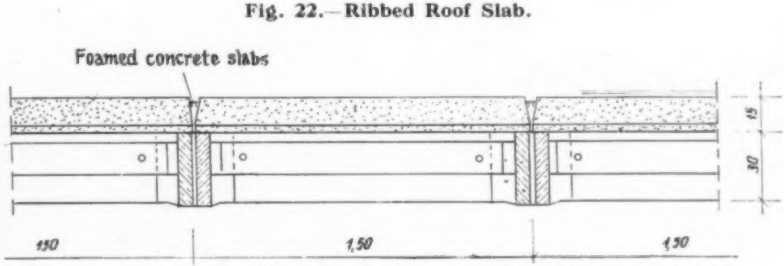


Fig. 23.—Foamed Concrete Roof Slabs.

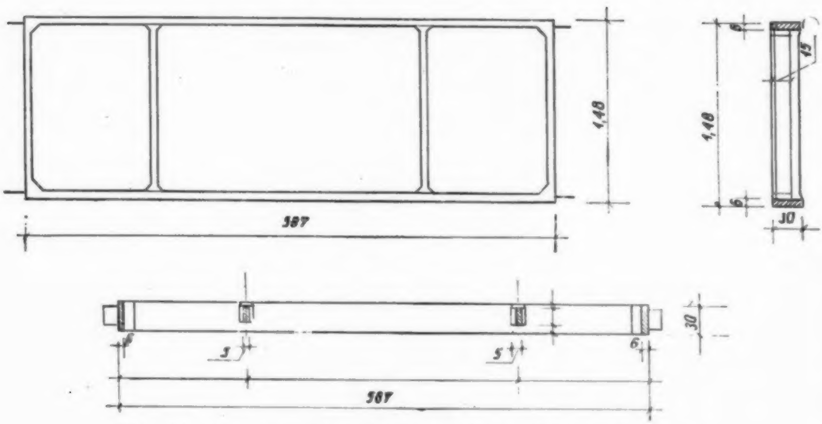


Fig. 24.—Framework to support Foamed Concrete Slabs.

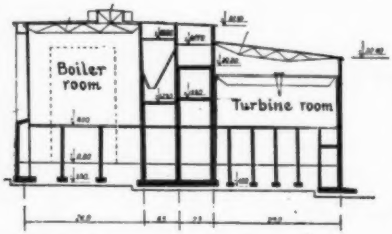


Fig. 25.—Cross Section through Power Station.

lined with sheet steel. They were erected by a crane composed of two masts 138 ft. high connected transversely. The column was lifted near the top, thus forming a cantilever beam during the erection (*Figs.* 26 and 27). The base was placed on a truck which was restrained by winches (*Figs.* 28 and 29). The bending moments due to the weight of the column were resisted by temporary prestressing. Since these moments decrease as the column approaches the vertical position the amount of the prestressing force should be variable, and this was obtained by attaching the ends of the prestressing cables to a winch which was provided with a dynamometer so that the force could be adjusted as required.

The column was anchored to the foundation by four rag-bolts 10 ft. long ; the base of the column was provided with a rolled-steel frame with protruding fittings with holes for the foundation bolts.

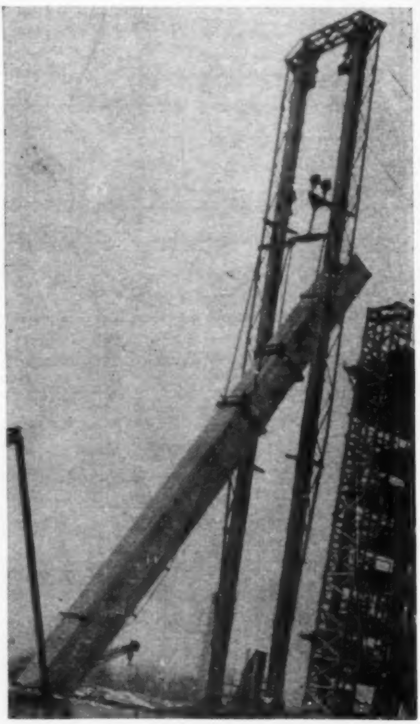


Fig. 26.—Hoisting a Precast Column.

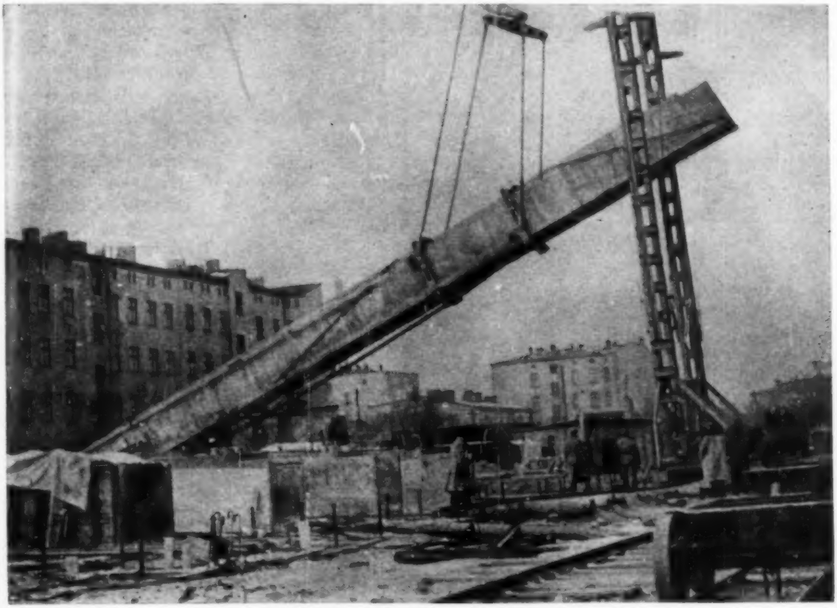


Fig. 27.—Hoisting a Precast Column.

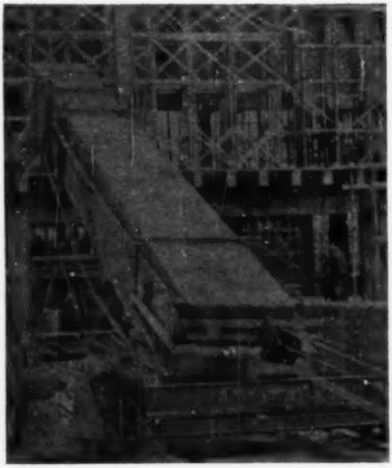


Fig. 28.—Base of a Column mounted on a Truck.

Fig. 29.—Method of Supporting the Base of a Column on a Truck.

The boiler-house and turbine-room are roofed with prestressed lattice girders with pre-tensioned wires of the type described previously. The other prefabricated elements included columns weighing 17½ tons (*Fig.* 31) for the turbine-room, prestressed crane girders of hollow section, cornices, wall slabs measuring 20 ft. by 16 ft., and windows. The middle part of the building was cast in place.

Another example of a prefabricated industrial structure is the burnt-clay containers at Jaroszów (designed by Mr. W. Zalewski and Mr. J. Draguła). Thirty-two containers with a capacity of 4800 cu. ft. are arranged in two rows of sixteen (*Figs.* 32 and 33). Each container is supported by two A-shaped structures, the upper parts of which are connected by transverse members (*Fig.* 34). A slight inclination of the sides reduces the bending moments in the wall of the container. On this structure the bottom of the container is built; the bottom is made in three parts, two of which form the hopper in the form of a truncated pyramid, and the third constitutes (together with the

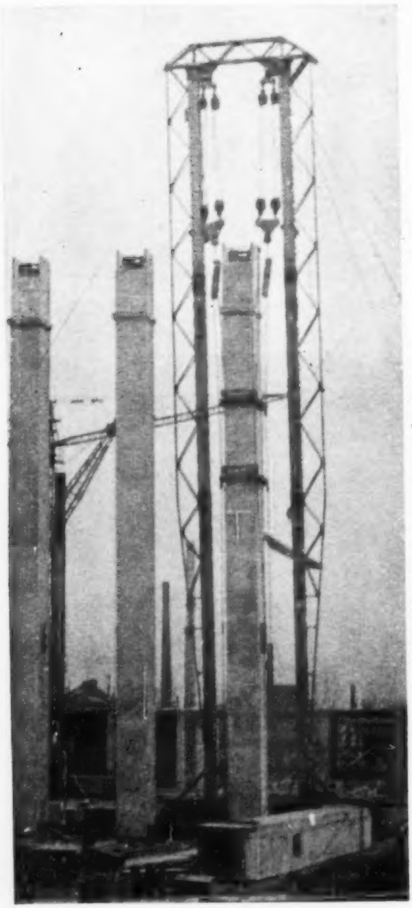


Fig. 30.—Precast Columns, 102 ft. High.

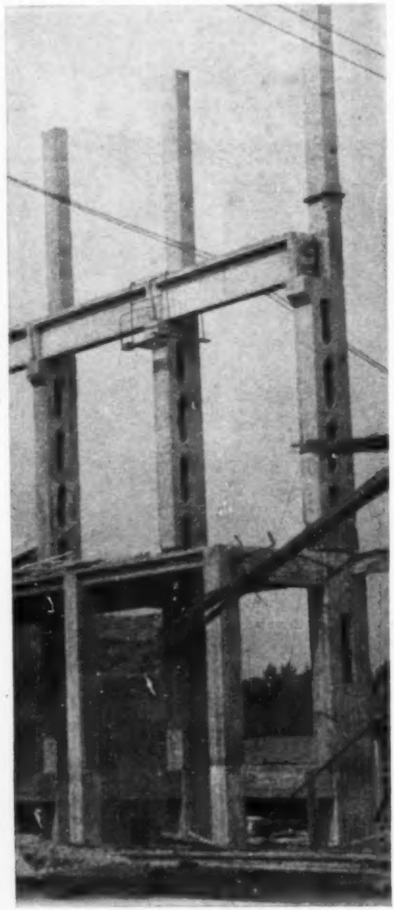


Fig. 31.—Precast Columns for a Turbine Room.

ribbed upper slab) the remaining part (*Fig.* 35). The containers support reinforced concrete frames 20 ft. apart and are covered by ribbed slabs. Belt-conveyors are installed below the frames to supply burnt clay to the containers.

The dimensions of the prefabricated elements were determined by the lifting capacity (20 tons) of the crane available. Each element of the hopper weighed about 17 tons, the supporting structure 14 tons, and the frames 10 tons each. The roof slabs weigh 1¼ tons each, the wall slabs 3 tons each, and the windows 2 tons. The precast elements were made on the site. The hopper elements were made in wooden moulds lined with sheet steel, using high-strength cement. The production cycle required 48 hours. The frames and the supporting structure were made in stacks, using sliding shutters, adjacent elements being separated by a lime-clay mortar brushed on to the frames already cast.

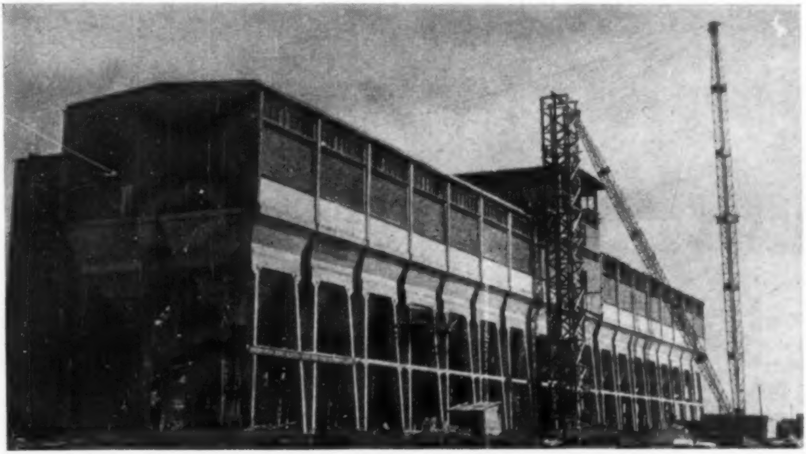


Fig. 32.—Prefabricated Containers.

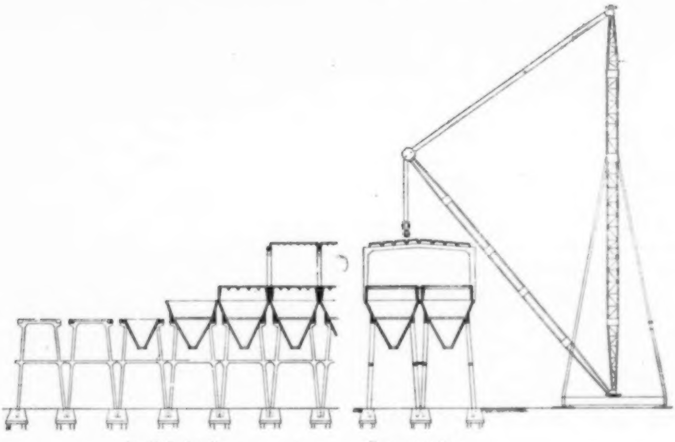


Fig. 33.—Arrangement of Containers and Supports.

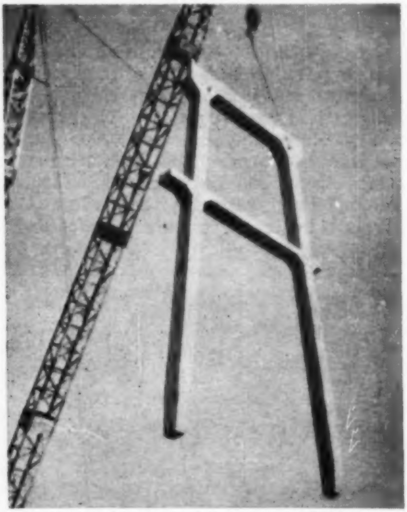


Fig. 34.—Frame for supporting the Containers.

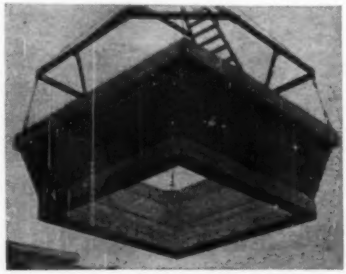


Fig. 35.—A Precast Element for a Container.

The joints were made by welding protruding steel strips or angles and filling the space with concrete. The use of precast elements, instead of the monolithic structure previously designed, resulted in the saving of 550 tons of steel, 31,500 cu. ft. of concrete, and 14,000 cu. ft. of wood.

The structure is nearly 600 ft. long, 42 ft. wide and 63 ft. high.

The photographs in this article are published by courtesy of the Department of Building Technique of the Ministry of Building, or were taken by Mr. Z. Zieliński and Mr. W. Drużdżel or by the authors. The drawings are from the publications of the Bureau of Standard Building Designs.

Fire Tests on Structural Members.

RESULTS of recent tests to determine the resistance to fire of several types of structural members are given in "Fire Research, 1956" * from which the following notes are abstracted.

INSULATED FLOOR.—A reinforced concrete floor 3½ in. thick, having a fire resistance when unprotected of less than half an hour, was coated with vermiculite-gypsum plaster 1 in. thick applied directly to the soffit as left by the shuttering. The construction provided a fire-resistance of 4 hours and although the plaster was cracked at the end of the test it was still firmly attached to the concrete over the whole of the soffit.

LIGHTWEIGHT AGGREGATE.—A structural steel column having a solid casing made with a mixture of "Perlite" lightweight aggregate and cement, sprayed on to a thickness of 2 in. withstood successfully a test of 6 hours without the temperature of the steel rising above 400 deg. C. (Failure of a column in the standard test is unlikely to occur until the mean temperature of the steel exceeds 550 deg. C.) A brick wall 4½ in. thick, finished on either side with Perlite-gypsum plaster ½ in. thick, satisfied the test requirements for a fire-resistance of 6 hours.

PRESTRESSED CONCRETE COLUMNS.—In every test of a prestressed concrete column when the heating was continued until failure occurred, the collapse of the column was sudden and complete. One column, which was still supporting its design load when the test was stopped, lost strength on cooling to such an extent that, when reloaded 48 hours later, failure occurred at 55 per cent. of the design load.

* Published by H.M.S.O. Price 4s.

North-Light Barrel-Vault Roofs, Grimsby.

By C. V. BLUMFIELD, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

THE roof of the manufacturing section of an extension of a factory at Grimsby consists of nineteen north-light barrel-vaults, each 80 ft. long and 31 ft. wide between the centres of the valleys.

In order to speed the work, precast concrete was used wherever practicable, including the end gable-frames and the valley beams. This allowed the end gable-frames to be placed against the wall of an existing building, and it was also possible to tension the steel in the prestressed valley beams from both ends. The end gable-frames were cast on the ground in one piece and lifted into position by a mobile crane (*Fig.* 1). The columns rested on prepared foundations, the top part of which was cast after the frames had been aligned and plumbed, so as to fix rigidly the feet of the frames. The weight of each frame was 14 tons, and in order to keep the weight within the capacity of the crane, the horizontal tie was made hollow by inserting a cardboard former.

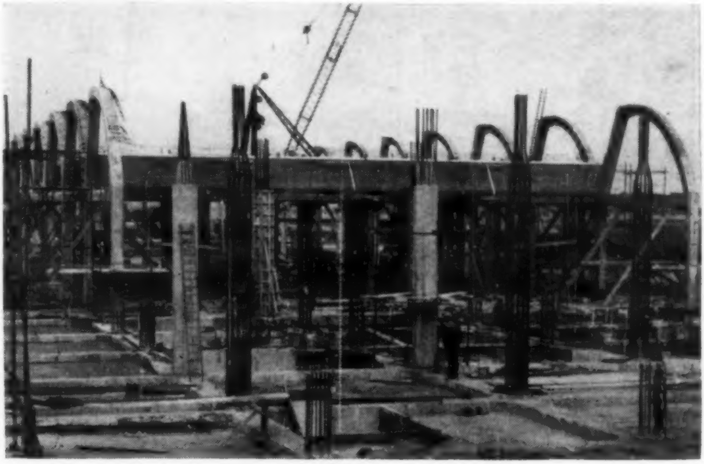


Fig. 1.

The V-shaped valley beams, which weighed about 38 tons each, were cast on shutters just above their final positions (*Fig.* 2). The ends of these beams, where the anchorage stresses caused by the forces in the prestressing cables are greatest, were precast, using concrete with a strength of 6000 lb. per square inch at the time the prestress was applied to the concrete. These end pieces were 4 ft. long, and were placed in the shutters before casting the valley beams, for which the minimum specified strength of the concrete was 3500 lb. per square inch at the time the prestress was applied to the concrete and 4500 lb. per square inch at 28 days. The beams were prestressed with six cables, each of twelve 0·276 in. wires, by means of the Gifford-Udall system, and were finally lowered on to the gable-frames, thus largely eliminating the bending moment on the frames which might have been caused by the shortening of the valley beams. The connections between the frames and the beams were made by dowel bars projecting from the frames. Where the ends of the valley beams were against the existing building the supports for the precast beams were arranged to give a clearance of 4 ft. between the ends of the beams and the existing wall; the beams were moved into their final positions on rollers after prestressing.

Following the removal of the shutters from the valley beam, the shuttering for

the 3-in. vault was fixed and the expanded-metal mesh which reinforces the shell (*Fig.* 3) was connected with similar mesh projecting from the valley beam.

This process of partial precasting speeded construction by allowing the shuttering for the vaults to be rapidly removed and re-erected, and by allowing the comparatively lengthy process of casting and prestressing the valley beams to proceed in advance of the construction of the vaults. One complete vault was cast every five days during most of the period.

The consulting engineer was Mr. Norman King, and the north-light barrel-vaults were designed by the writer. The general contractors were Sir Robert McAlpine & Sons, Ltd.

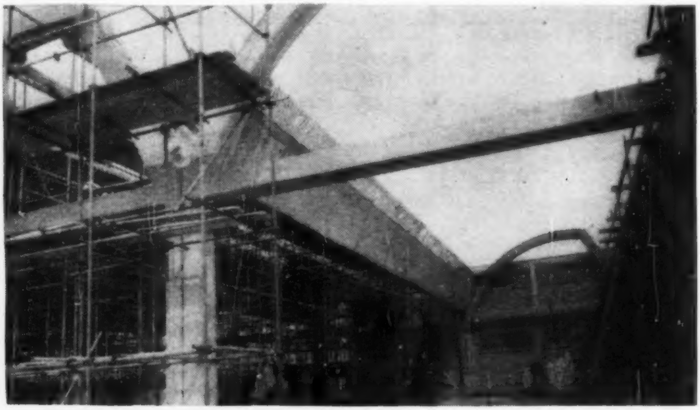


Fig. 2.

Fig. 3.

An Hotel and Coach Station in Dover.

AN hotel and coach station (*Fig.* 1) recently constructed at Dover is in two parts, namely a single-story building of timber construction accommodating a restaurant, ballroom, and bars, and a reinforced concrete structure 65 ft. high containing bedrooms which are at some distance from the restaurant so that no noise is transmitted to them. The gable of this building, which is 22 ft. wide, projects beyond the building line of the timber structure and faces towards the sea. Sections are given in *Fig.* 3.

The reinforced concrete building stands on a platform 22 ft. above the level of the street. The platform is carried on four V-shaped supports, each leg of the V being 18 in. square in cross section. Each of the two longitudinal beams at the level of the first floor, with its V-shaped supports and ties below ground level, forms a statically-indeterminate structure with one redundant (*Fig.* 2). A hinge is provided between each leg of these supports and the beams in order to avoid stresses due to bending in the columns. The beams, which carry five floors and the roof, are 4 ft. 6 in. deep by 1 ft. 6 in. wide and taper to 3 ft. 6 in. at the ends of the cantilevers. The floors and the roof form a framed structure, consisting of 12-in. by 12-in. columns, 12-in. by 12-in. longitudinal beams, and 18-in. by 9-in. transverse beams.

The floors and roof are of hollow-tile construction, 6 in. thick, and span 10 ft. 6 in. On one side of the building balconies on each floor cantilever 6 ft.; they increase the available floor space and allow a view of the sea from nearly all the bedrooms. A lift and a reinforced concrete staircase with timber treads provide access to the upper floors from the foyer on the ground floor. An escape staircase between the third and fifth floors only is situated at the south-west corner of the building. There is a mezzanine floor, carried directly on the V-shaped supports, over part of the ground floor.

Fig. 1.

All external faces of the structure were treated with a chemical in order to expose the aggregate.

The top 10 ft. of the ground were made

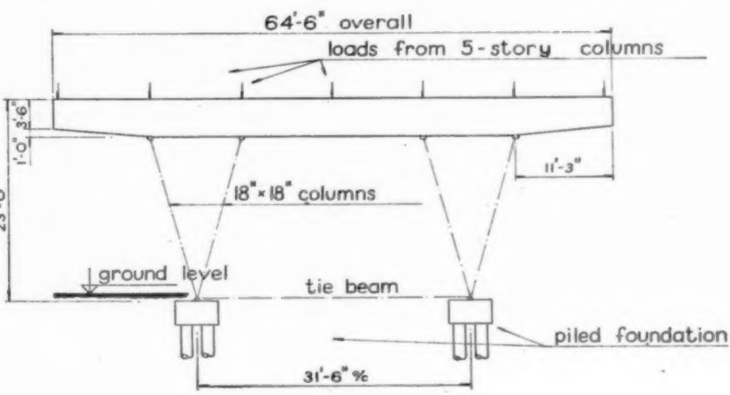


Fig. 2.

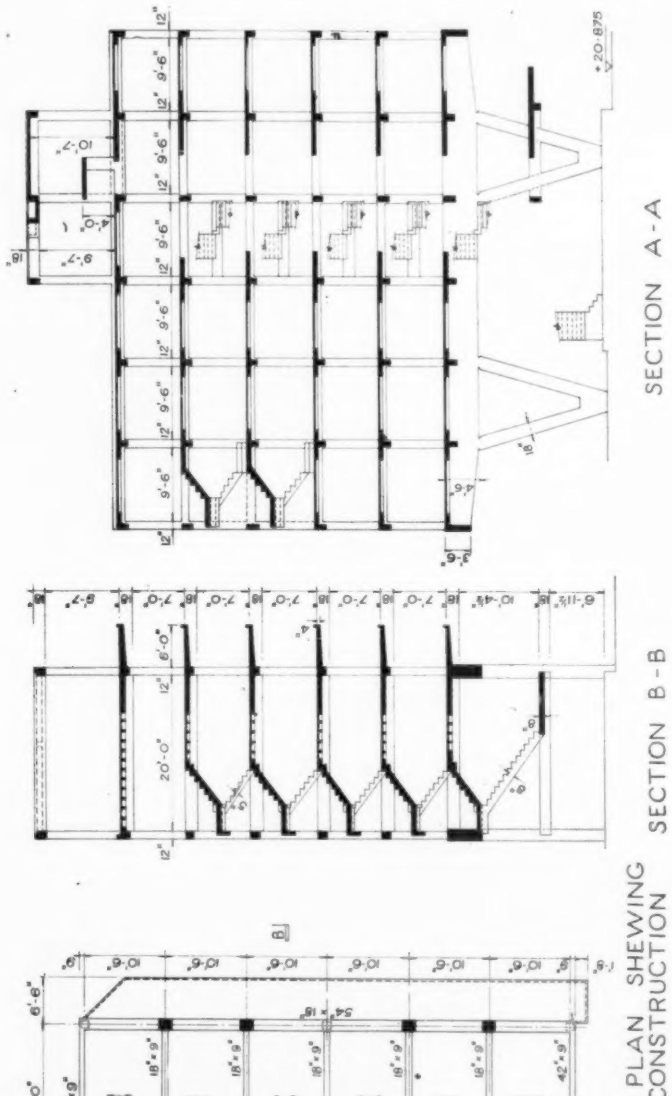

SECTION A - A
SECTION B - B
MEZZANINE PLAN SHEWING
1st. FLOOR CONSTRUCTION

Fig. 3.

up of brick rubble, with a very small load-bearing capacity, filled into the basements of buildings that previously stood on the site. Twenty-two bored piles, each penetrating about 40 ft. into the ground, carry the concentrated loads from the larger building; the greatest load on a pile is 60 tons. The comparatively small and widespread loads from the timber building are supported by continuous strip footings varying between 3 ft. and 5 ft. in width; the pressure on the ground below these footings is limited to $\frac{1}{3}$ ton per square foot. The north-west corner of the timber structure overhangs a steep bank of a stream; this part of the ground floor is suspended and supported on three walls built on strip foundations 15 ft. below the level of the floor. The concrete and timber structures are completely separated by expansion joints so as to avoid damage due to differential settlement.

The architect was Mr. Louis Erdi, the consulting engineer was Dr. K. Hajnal-Kónyi and the contractor was Mr. R. J. Barwick.

Extension of a Hospital.

An eight-story extension to the Leeds General Infirmary was designed using a 3 ft. 2 in. longitudinal module. The external columns are at 22 ft. 2 in. centres. On each floor there are two wards, 21 ft. and 16 ft. wide, with a corridor 7 ft. 6 in. wide between them. The height between each story is 11 ft. in conformity with the floor levels of the existing building.

Fig. 1 shows the floor construction. The thickness is 1 ft. 5 in., comprising structural concrete 9 in. thick, a finish 2 in. thick, and a false ceiling 6 in. thick. Solid parts at the column-heads resist the shearing forces. The average weight of the floor, including the solid parts, is 58 lb. per square foot, and the weight of the coffered section alone is $45\frac{1}{2}$ lb. per square foot. The thickness of the structural slab in this part is $1\frac{1}{2}$ in.; the ribs are $7\frac{1}{2}$ in. deep and taper from $4\frac{1}{2}$ in. to $3\frac{1}{2}$ in. in width. This form of construction was found to be more economical than a solid flat slab. Metal pans were used to form the coffers in the slab.

The external columns are 3 ft. 2 in. by 6 in. in cross section. The gable walls and lift shafts are in reinforced concrete, and give additional stability to the structure.

The architects are Messrs. Kitson, Parish, Ledgard, and Pyman, the consulting engineers are Messrs. Ove Arup & Partners, and the contractors are Messrs. J. Gerrard & Sons, Ltd.

Fig. 1.

A Church at Hanworth.

A LONGITUDINAL section through the Church of All Saints, Hanworth, Middlesex, is shown in *Fig.* 1. The nave is about 50 ft. square and 35 ft. high, and is surmounted by a lantern 24 ft. in diameter and a cross. Two slender reinforced concrete arches, intersecting at the centre, span diagonally across the nave and support the roof and the lantern by means of a ring-beam. The roof is of beam-and-slab construction, the beams spanning diagonally from the ring-beam to the brick walls of the nave.

Projecting from the sides of the nave are the transepts, vestries, and the apse. The apse is formed by a reinforced concrete "shell", 2½ in. thick, the inner surface of which is in the shape of a quarter-sphere, the centre being 2 ft. 3 in. above floor level. The inner edge of the apse is thickened to form an arch, which supports the brick wall of the nave.

The lantern consists of four reinforced concrete columns, which stand on the ring-beam, and which carry the roof and twenty intermediate precast mullions.

The church was completed in 1957 at a cost of £42,000. The architect was Mr. N. F. Cachemaille-Day, the reinforced concrete was designed by G.K.N. Reinforcements, Ltd., and the contractors were Messrs. Haymills (Contractors), Ltd.

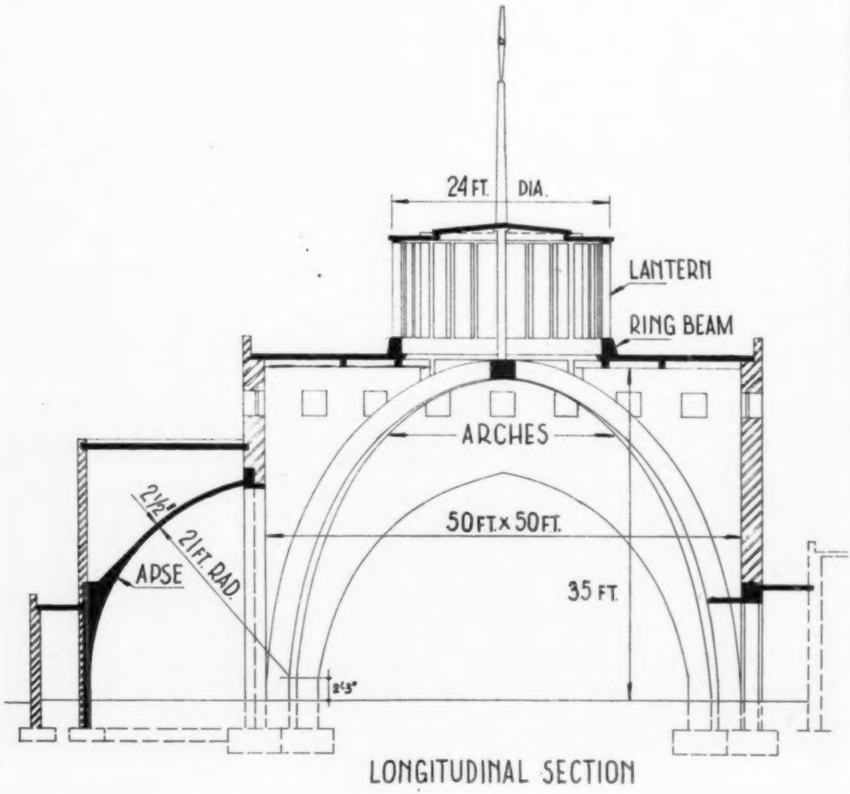


Fig. 1.

An Unusual Roof.

By J. O. KALNINS, A.M.I.Struct.E.

THE new council chamber at Warwick is a many-sided structure, the plan of which is roughly semicircular. The roof consists of two slabs, supported by Vierendeel girders which rest on the load-bearing brick walls of the chamber. *Fig.* 1 shows the underside of the completed roof, and *Figs.* 2 and 3 show the lower and upper roof slabs respectively.

The girders radiate from a central point to the walls, the longest span being about 75 ft. Before the design was prepared, a balsa-wood model of the structure was used to determine the most efficient arrangement of the girders. The bottom chords of the girders were prestressed by the P.S.C. system. Each prestressing cable consists of four 0·276-in. high-tensile steel wires surrounded by ⅞-in. square metal sheaths, which prevented the wires from changing their relative positions and also reduced the friction which would otherwise have occurred between the concrete and the wires. The top chords are reinforced with mild steel bars ; with the exception of the ring-beams (*Fig.* 4), no prestress was applied to these chords.

The secondary beams and slabs of the

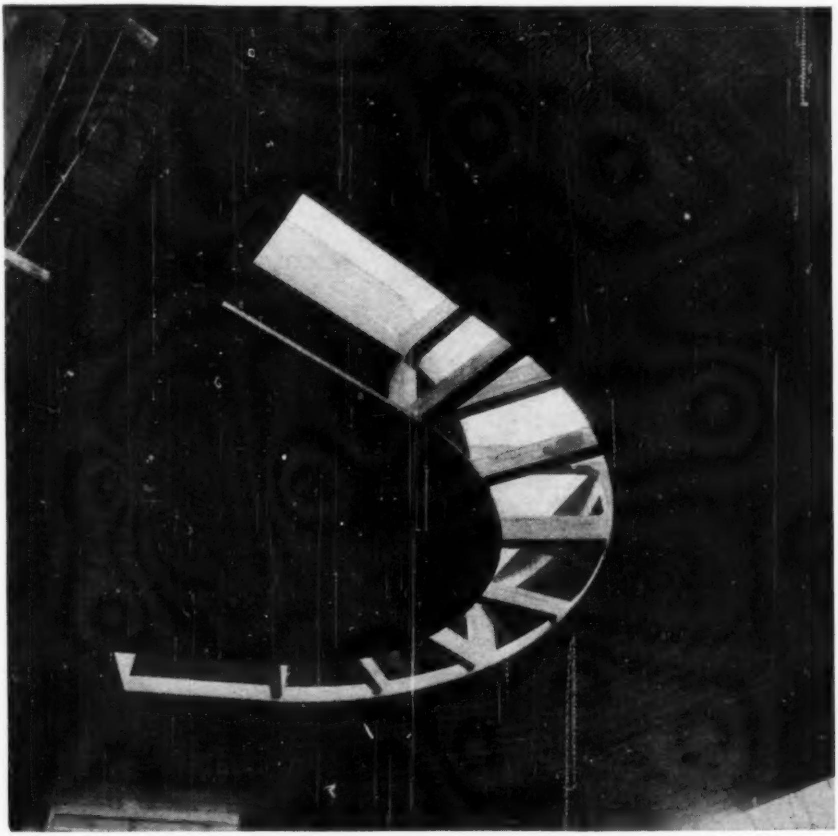


Fig. 1.

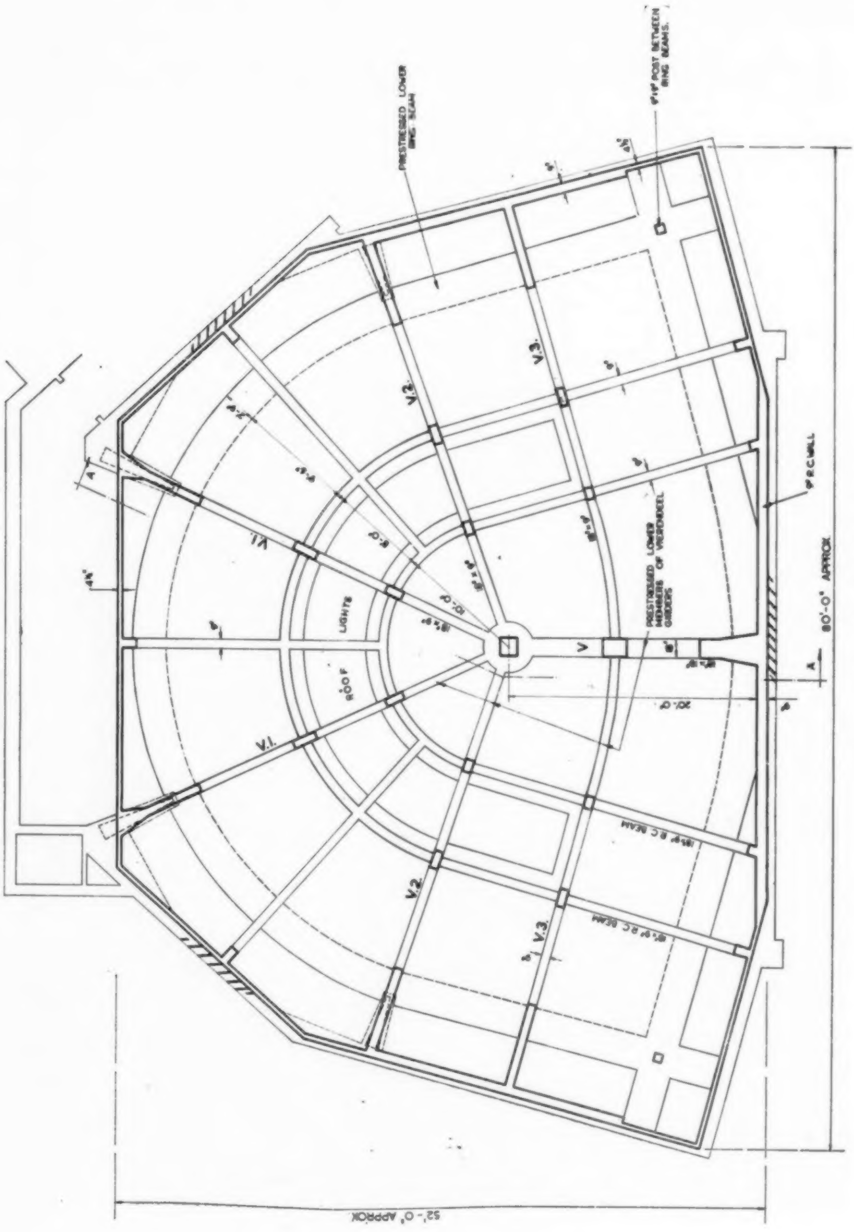


Fig. 2.—Lower Roof Slabs.

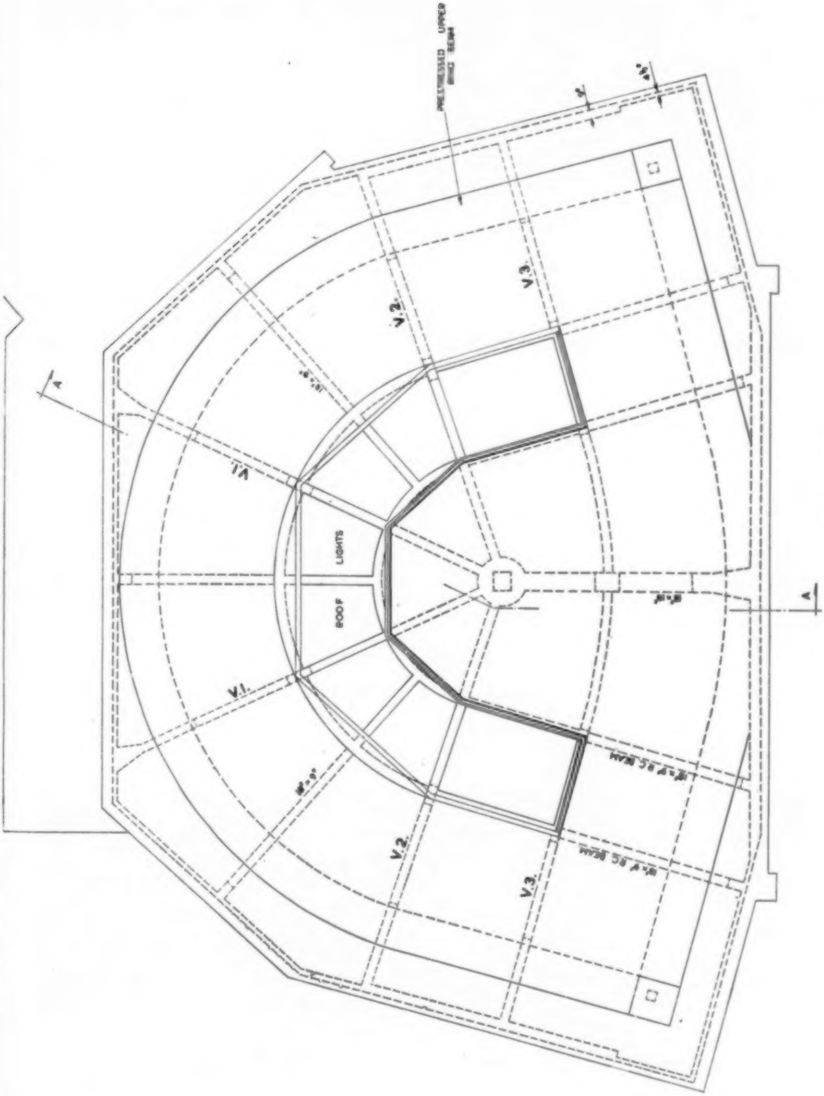


Fig. 2.—Lower Roof Slabs.

Fig. 3.—Upper Roof Slabs.

roof are supported by the top and bottom chords, and the distance of 6 ft. between them permits access to the services. There is a glazed opening in the shape of a horse-shoe (*Fig.* 3) in the upper and lower slabs, and artificial lighting, ventilation, and acoustical treatment are incorporated in the ceiling. The centre of the roof is 2 ft. above the edges in order to produce a domed effect; to counteract the horizontal thrust of the dome, upper and lower prestressed ring-beams are provided.

Design of Girders.

The Vierendeel girders were analysed by the deformation method,* and the calculated deflections were compared with those observed. Each girder has only computing machine. "Exact" values for the deflections at the junctions were then calculated by introducing these bending moments into the original equations. All shearing and axial forces were calculated, and prestressing forces were introduced at the eccentricities necessary to counteract the tensions produced by the combined action of the axial forces and the bending moments.

Mild steel reinforcement was necessary at some junctions where reversal of bending moments occurred (*Fig.* 6). A few members subject to exceptionally high shearing stresses were reinforced by welded mild steel cages of diagonal and longitudinal bars.

The calculations showed that the influence of continuity on girders V.2 and V.3

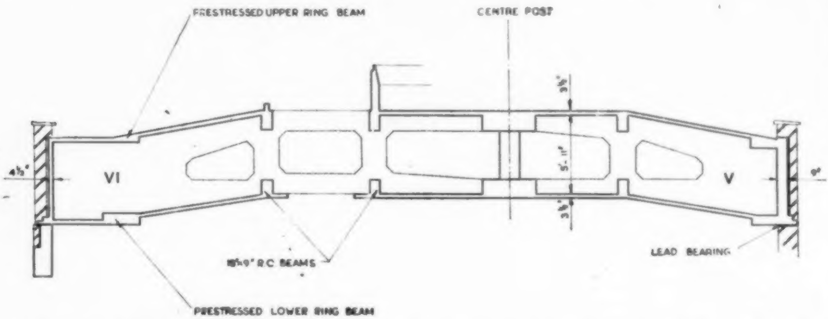


Fig. 4.—Main Vierendeel Girder V/V.1 in Elevation (Section AA on Fig. 2).

one axis of symmetry, nearly every member is of different size and shape (*Fig.* 5), and each junction carries a different load. A comparatively accurate analysis was, however, possible. The stiffness of the main girder V.1/V is considerably greater than the stiffnesses of girders V.2 and V.3 (which are also curved on plan), and it was therefore assumed that these girders would be supported by girder V.1 and would develop full continuity over it. All junctions were considered to be released, and equations of equilibrium were obtained for the simultaneous closure of all the gaps.

The maximum number of simultaneous equations was ten (for girder V.1/V), and these were solved for the statically-indeterminate moments by means of a was comparatively small. The greatest calculated deflection, based on a modulus of elasticity of 4,000,000 lb. per square inch, was 0·2 in., and the corresponding observed deflection was ⅛ in., indicating that the modulus of elasticity of the concrete exceeded that assumed.

Sequence of Construction.

The construction was carried out as follows.

1.—After the erection of the scaffolding and the shuttering for the lower slab, the mild steel reinforcement was fixed for the bottom chords of the Vierendeel girders, the lower ring-beams, and the secondary beams. The prestressing cables and welded reinforcement were then placed in position.

2.—The bottom chords of girders type V.1/V were cast, and the steel was

* 'Analysis of Structures', by M. Smolira. Concrete Publications, Ltd.

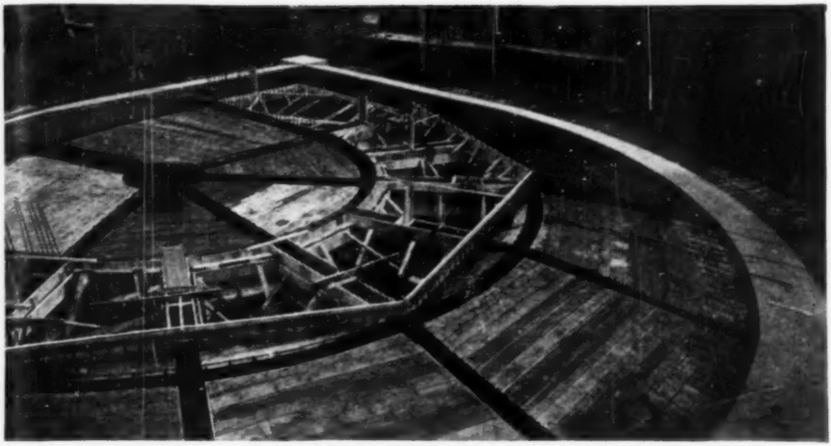


Fig. 5.

tensioned when the crushing strength of concrete cubes was at least 5000 lb. per square inch.

3.—The bottom chords of girders V.2 and V.3 were cast and prestressed.

4.—The lower ring-beam was concreted and prestressed. In this case the cable was tensioned from both ends to avoid excessive losses due to friction.

5.—All the lower secondary beams and the $3\frac{1}{2}$-in. slab were cast, using combined reinforcement and centering in order to save timber and to obtain a good surface for plastering. The posts of all the girders were cast up to the level of the soffits of the upper beams.

6.—The reinforcement for the upper beams, the perimetral walls, and the upper ring-beam was fixed, and the prestressing cables were arranged in position.

7.—The upper ring-beam was cast and prestressed.

8.—The perimetral walls were cast in two lifts.

9.—The shuttering was erected and the reinforcement fixed for the upper roof-slab.

10.—The top chords of the girders and the secondary beams were cast monolithically with the $3\frac{1}{2}$-in. slab.

11.—All the props were removed. The maximum deflection measured at the centre of the roof was about $\frac{1}{8}$ in.

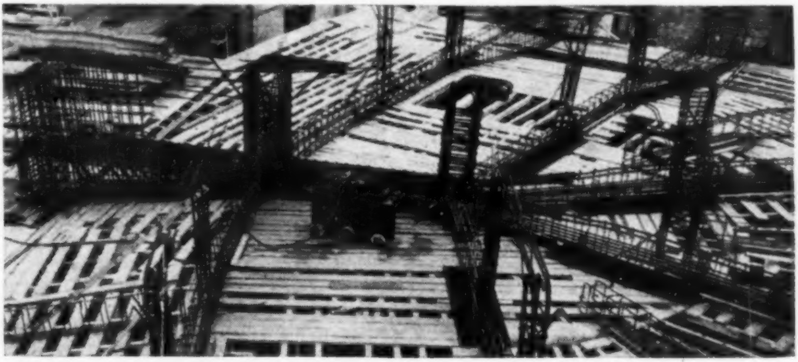


Fig. 6.—Lower Chords of Vierendeel Girders : Mild Steel Reinforcement and Prestressing Cables in Position.

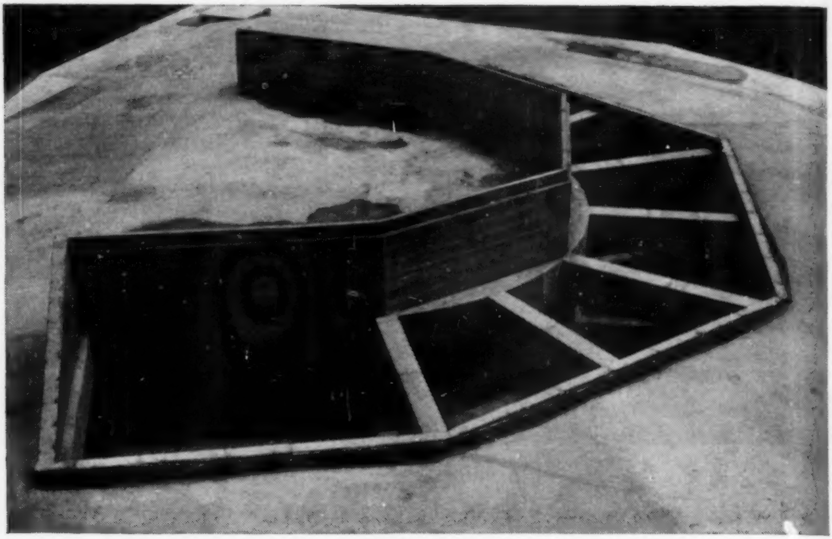


Fig. 7.

The required crushing strength of concrete cubes for the prestressed work was 6000 lb. per square inch at 28 days, but the actual average cube strength exceeded 7000 lb. per square inch. The ingredients of the concrete were proportioned by weight and the concrete was consolidated by internal vibrators.

When all the prestressing cables had been tensioned, each cable was grouted from one end. The operation was continued until grout appeared at the holes at the other end of each member. The cables, which were 120 ft. long and curved on plan, were grouted from one end without difficulty.

A view from above of the completed roof is shown in *Fig. 7*.

The County Architect is Mr. G. R. Barnsley, F.R.I.B.A., the general contractors were Messrs. Foster & Dicksee, Ltd., and the design and construction of the reinforced concrete were by the Trussed Concrete Steel Co., Ltd.

Standard Design for Warehouses in the U.S.A.

A TYPE of construction that has been standardised by the U.S. Corps of Engineers is being used for two single-story structures, one 2000 ft. long by 400 ft. wide and the other 1800 ft. long by 400 ft. wide, for the storage of aircraft parts at an American Air Force base in California.

The rectangular frames are cast in place; these have main beams, supported on columns, continuous across the width of the buildings. The secondary beams are precast. The floor is 8 in. thick and is not reinforced. The exterior and interior walls are cast on the floor and hoisted into position; some of the slabs for the internal fire-resisting walls are 27 ft. by 24 ft. by 8 in. thick and weigh 27 tons.

The roof slabs are precast in steel moulds. They measure 33 ft. 4 in. by 5 ft., and are 1 ft. thick at the ribs and $1\frac{1}{4}$ in. thick at the centre. These slabs are made with expanded-shale aggregate, with a topping of 2 in. of insulating concrete and a covering of asphalt.

An Unusual Water Tower.

INCREASING production on a limited site necessitated the construction of the combined water-clarifying tower and storage tank, 40 ft. diameter and 90 ft. high, shown in *Figs.* 1 and 2, in which a reverse annular-flow filter of 300,000 gallons capacity (to the clients' design) supports a covered tank of 60,000 gallons capacity. Provision is made for the future construction of a second tower.

The upper storage tank is annular. The agitating gear for the filter is in the inner cylinder and the tank is supported by 12 columns which stand on the inner and outer walls of the clarifying tower. The outer wall also carries cantilevered collection and overflow troughs, a precast concrete walkway and handrail standards.

The inner and outer walls of the tower both rest on a circular beam and cantilevered brackets supported on eight columns 3 ft. square. These columns, carrying a total load of 3500 tons, are on a pitch circle of 28 ft. diameter to clear an adjoining road and are supported on 88 bored piles. The level of the bottom of the conical hopper was determined by the pressure-head required to cause suspended matter accumulating in the hopper to flow freely. The brackets and supporting columns are shown in *Figs.* 2 and 3.

The design and constructional details are based on the Code of Practice for the Design and Construction of Reinforced Concrete Structures for the Storage of Liquids issued by the Institution of Civil Engineers, and on the American Portland Cement Association's publication on Circular Concrete Tanks; the permissible

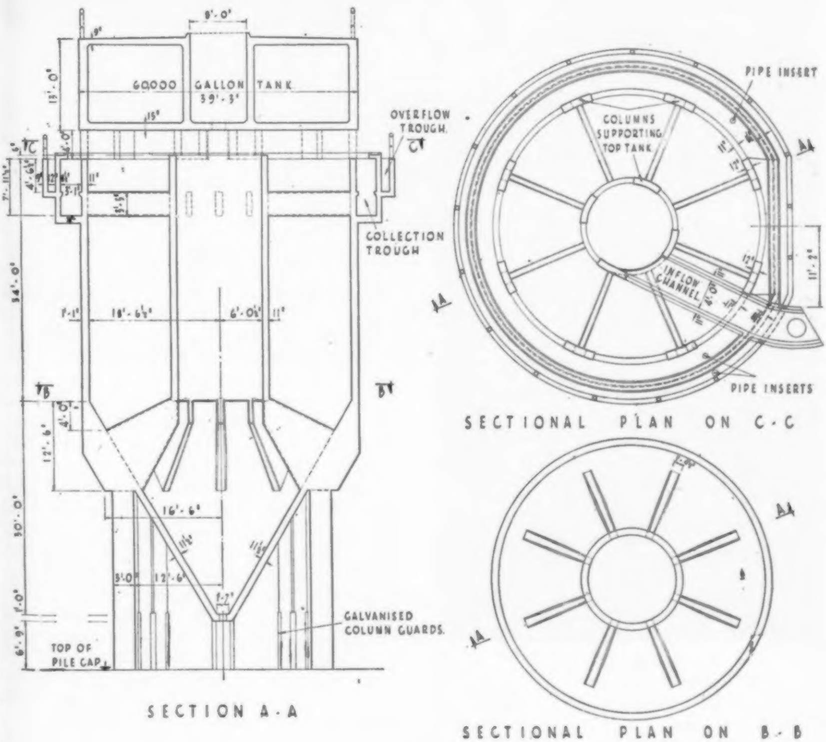


Fig. 1

stresses were modified to allow for the use of high-grade concrete. The design of the walls of the chamber allowed for the stresses due to the temperature of the contents, which was assumed to be 100 deg. Fahr. The stresses in the ring-beam due to temperature were investigated by the method indicated by Mr. S. W. Lewaren in his article on "The Design of Reinforced Concrete Pressure Vessels" (see this journal for February, 1950).

All the construction joints are horizontal and include a 6-in. P.V.C. water-bar. A few hours after placing the concrete the top surface of each lift of

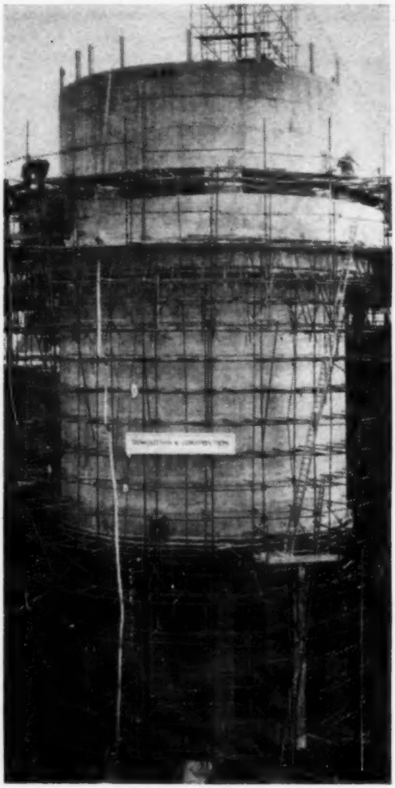


Fig. 2.

Fig. 3.

4 ft. was sprayed with water under high pressure to remove the laitance. Provision was made for curing the outer wall by continuously spraying it with water. After the storage tank and filter had been tested for watertightness the filter was lined with cement-sand rendering to reduce accretions of solids.

The proportions of the constituents of the concrete were 1 : 1·6 : 3·2, with coarse aggregates graded from $\frac{3}{4}$ in. to $\frac{3}{8}$ in. and $\frac{3}{8}$ in. to $\frac{3}{16}$ in. Consolidation was by internal vibration. The specified minimum crushing strength of 6-in. cubes was 4000 lb. per square inch at 28 days, and the average crushing strength obtained was 5500 lb. per square inch at 28 days. The compacting factor was 0·84.

The tower is at the works of The New Merton Board Mills, Ltd. The general contractors were the Demolition & Construction Co., Ltd., the bored piling was carried out by the Pressure Piling Co., Ltd., and the lining to the filter was by Messrs. James Scott & Son, Ltd. The consulting engineers were Messrs. John F. Farquharson & Partners.

Prestressed Tanks at a Sewage Works.

THE tanks described are part of the Rye Meads Sewage Purification Works now being built near Hoddesdon, Herts, for the Harlow and Stevenage Development Corporation in connection with the Middle Lee main drainage scheme. The tanks will provide storage capacity for sludge between the continuous process of digestion and the subsequent air drying on open beds, which is a seasonal operation. Four tanks have been constructed, each with a capacity of about 1,500,000 gallons. Each has an internal diameter of 100 ft., and the maximum depth of water contained is 31 ft. The walls are of prestressed concrete with a uniform thickness of 8 in. and are provided with a stiffening ring 3 ft. 11 in. wide at the top which also acts as a walkway where required.

The floors of the tanks (*Fig.* 1), which have a fall of 12 in. towards the centre, are of reinforced concrete and are designed in accordance with the normal requirements for water-retaining structures. Sludge enters each tank through a pipe of 9-in. diameter, which passes through the wall 1 ft. 3 in. above floor level, and is drawn off from the centre at floor level through a pipe incorporated in the floor. There is also a 4-in. water draw-off pipe passing through the wall 19 ft. above floor level.

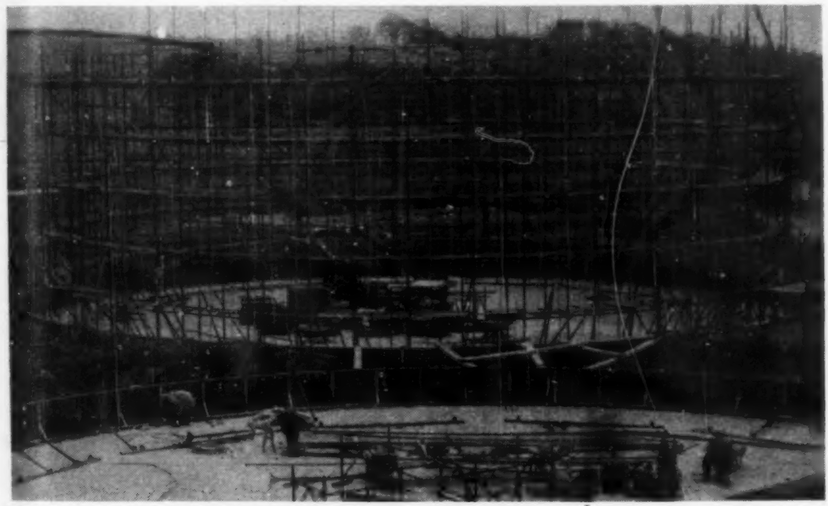


Fig. 1.

Construction.

A sliding joint (*Fig.* 3), formed between the wall and the floor, was sealed with bituminous compound. As a further precaution against leakage a P.V.C. waterstop 6 in. wide is incorporated in the joint. Steps were taken to prevent this strip from shearing as a result of the movement of the wall due to horizontal prestressing.

The walls were prestressed vertically and horizontally. The vertical prestressing (*Figs.* 4 and 5) was effected by means of $\frac{7}{8}$-in. Macalloy bars at 2 ft. 4 in. centres in the middle of the wall, and the horizontal prestressing (*Figs.* 2 and 6) by cables, each composed of four 0·276-in. high-tensile wires anchored in Magnel-Blaton sandwich-plates. The spacing of the cables varies from $2\frac{1}{4}$ in. at the base of the wall to 16 in. at the top. The vertical bars were tensioned before the horizontal cables. This ensured that the temporary stresses set up in the wall of the tank during the horizontal prestressing did not exceed permissible limits.

Each lift of the wall was cast in a continuous operation, the first lift being 12 in. and each subsequent lift 3 ft. 6 in. A few hours after casting, all scum was removed by jetting with a high-pressure hose and wire brushing. A layer of mortar was spread on the surface of the joint immediately before placing fresh concrete, which was then worked and vibrated into the mortar. Immersion vibrators were used throughout. The horizontal construction joints between adjacent lifts were not provided with water-bars.

After the first lift had been placed the prestressing bars, which were contained in flexible conduits 2¼ in. diameter, were fixed in position and the conduits built in as concreting proceeded. When the vertical stressing was completed colloidal grout was injected into the conduit through a steel tube attached to the conduit at a point close to the lower anchor-plate. A small groove was formed in the mortar-pad cast at the top of the wall for seating the anchorage for the prestressing bars. This groove provided a means of escape for the air driven out by the grout, which was allowed to escape also for a few seconds to ensure that the duct had been completely filled.

The stiffening-ring at the top of the wall was not constructed until after the completion of the prestressing operations, first because the greater stiffness of the ring compared with the wall might cause shearing of the construction joint, and secondly because the manipulation of the prestressing jack would have been impeded.

The horizontal cables were supported prior to tensioning by 1 in. by 16-gauge mild steel strips notched to ensure that the cables were correctly spaced. Each cable was held away from the wall by ⅜-in. mild steel bars fixed vertically to the face of the wall at about 3 ft. centres. The main purpose of these bars was to reduce the friction between the horizontal cables and the wall.

On completion of the horizontal prestressing a light steel mesh was fixed to the cables and mortar, providing a minimum cover of 1¼ in., was applied pneumatically over the whole outer surface of the wall to protect the wires and anchorages. In order to prevent the development of tensile stresses in the mortar due to the outward movement of the wall under load, the mortar was applied after the tank had been filled. The fact that the horizontal cables were held away from the wall by the ⅜-in. bars enabled some of the mortar to penetrate behind the cables.

The inner face of the wall was cured with a sealing membrane, but hessian was used for the outer face. The sealing membrane was not used on the outer face as the bond between the mortar and the wall might have been affected by the continued adhesion of particles of the membrane to the wall.

At a low level a cast-iron pipe of 9 in. diameter was built into the wall. Owing to the close spacing of the cables it was

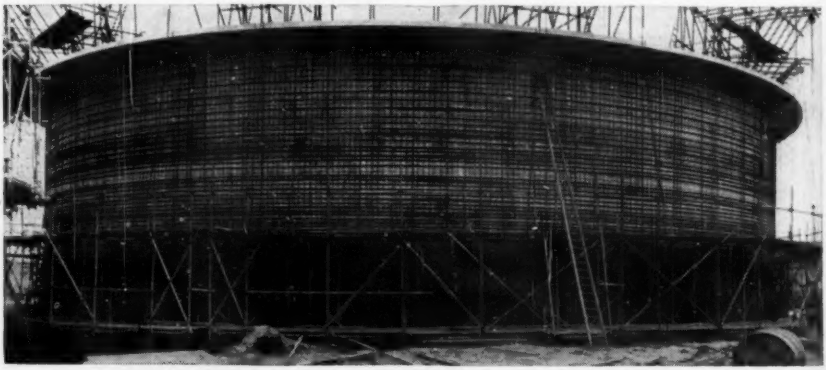


Fig. 2.

necessary to provide a welded steel frame in order to maintain continuity. The vertical members of the frame were each fitted with three steel pins of 2 in. diameter spaced vertically at 3½ in. centres. Two cables were looped around each upper and lower pin and one cable around the middle pin. Tests on samples of the wire bent to this radius showed a reduction in ultimate strength of about 10 per cent. Higher up the wall another cast-iron pipe passed through but here the spacing of the cables was wide enough to avoid the pipe and no frame was necessary.

Normal Portland cement was used throughout and the concrete was designed

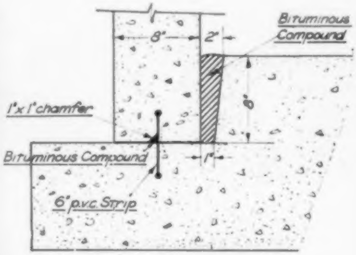


Fig. 3.

Fig. 4.

Fig. 5.

to have a minimum cube strength of 6000 lb. per square inch at 28 days. The water-cement ratio was 0·38 and the materials for each batch were weighed. The ratio of combined aggregate to cement was 3 : 1.

Design.

The formula used for determining the required vertical force was

$$P = 0{\cdot}275\,\frac{1 + \frac{3}{8}\,m.r}{1 + \frac{1}{2}\,m.r}.P_0 - d.F_{ct}$$

in which P_0 is the initial force in the horizontal cables per foot of height, m is the modular ratio (assumed to be equal to 6),

r is the $\dfrac{\text{area of steel per foot of height}}{\text{thickness of wall}}$,

d is the thickness of the wall, F_{ct} is the

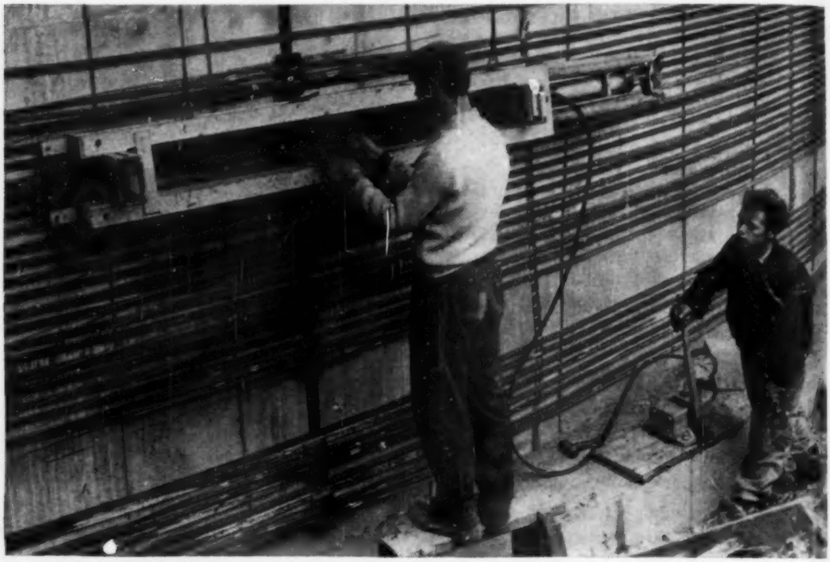


Fig. 6.

allowable tensile stress in the concrete in bending, and P is the vertical force required per foot of circumference.

The effective lengths of the horizontal cables were determined by the method suggested by Mr. E. H. Cooley; the coefficient of friction was assumed to be 0·35, but this was found to be high. Extensions were calculated from a datum corresponding to a gauge reading of 2500 lb. It was assumed that after 2500 lb. had been applied all the "slack" would be removed and any further movement would be due to elastic extension of the cable.

The stresses used in the calculations were (in lb. per square inch):

Maximum direct compressive stress in concrete, 1500.

Minimum direct compressive stress in concrete, 100.

Maximum tensile stress in bending in concrete, 250.

Young's Modulus for concrete at the time the prestress was applied to the concrete, 5×10^6.

Young's Modulus for concrete under working load, 3×10^6.

Maximum stress in steel, 140,000.

Modular ratio at the time the prestress was applied to the concrete, 6.

The loss of prestress due to shrinkage and creep was assumed to be 20 per cent. of the initial prestress, and the loss due to friction was calculated to be 24 per cent., assuming a coefficient of friction of 0·35.

The value of 0·35 for the coefficient of

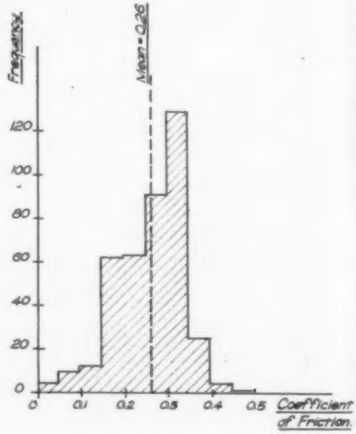


Fig. 7.

friction was found to be rather high, as was indicated by the fact that the calculated extension of the cable was in most cases reached before the calculated gauge-pressure. Since the average stress in the wall of the tank depends upon the extension of the cable rather than the gauge-reading, tensioning of the steel was discontinued when the calculated extension was reached; the steel was not therefore used at its maximum allowable stress. The final gauge-pressure for each stressing operation was noted, and from it the value of the coefficient of friction was determined. The results obtained are given in *Fig.* 7.

The joint consulting engineers for the Middle Lee Drainage Scheme are Messrs. J. D. & D. M. Watson and Messrs. D. Balfour & Sons. The main contractors for the prestressed tanks were Messrs. W. & C. French, Ltd.

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering"

January–February, 1908.

Reinforced Concrete Towers.—The Lincoln Light & Power Company have recently erected in the Province of Ontario reinforced concrete towers which are the highest monoliths in existence. These towers are 150 ft. long and 142 ft. above ground. They are 31 in. square at the base and 11 in. square at the top, and carry cross-arms for the electric service wires. The towers were moulded in a horizontal position, and then raised into place by wooden shear legs after being allowed to harden for more than a month. The concrete was composed of one part of Portland cement and five parts of sand and gravel, and four steel reinforcing rods, one at each corner, resist the tensile stresses due to wind pressure and pull of the wires.

L. G. Mouchel & Partners, Ltd.—The announcement has been made that Mr. L. G. Mouchel, the agent for the Hennebique system in this country, will henceforth conduct his work under the above description, the capital of the company being £50,000 in £1 shares. The first directors are Mr. L. G. Mouchel, J. S. E. De Vesian, and Mr. A. T. J. Gueritte, the remuneration of the directors being, except the managing director, £1800 per annum, 5 per cent. of the gross profits, and any further remuneration the shareholders may vote. The object of the company is described as being to carry on the business of specialists in "ferro-concrete" construction, engineers, architects, etc.

Systems.—An article entitled "Characteristics of the Chief Systems of Reinforced Concrete Applied to Buildings in Great Britain" describes ten systems for the design and construction of reinforced concrete beams, twenty for floors, eight for columns, eight for walls, one for chimneys, eight for piles, and eight for pipes.

[Note: "Concrete and Constructional Engineering" was published in alternate months until September, 1909, when monthly publication started.]

Arch Bridge of 951 ft. Span.

The Brazilian National Highway Department has awarded a contract for the construction of a reinforced concrete bridge across the Parana River, between Brazil and Paraguay. The overall length of the bridge will be between 1650 ft. and 2000 ft., and the main span of 951 ft. will be the longest open-spandrel reinforced concrete arch in the world. The deck will be 43 ft. wide, and its height above the average level of the river will be about 200 ft.

A Warehouse with Continuous Precast Frames.

THE framework of a warehouse recently built for Messrs. Charles Mason & Co., of Chesterfield, consists of five single-story continuous frames at 20 ft. centres, each of which comprises four north-light bays of 30 ft. span (*Fig.* 1), and one double-pitched frame of 45 ft. span (*Fig.* 2). The area of the warehouse is 165 ft. by 120 ft., the height to the eaves of the north-light frames is 10 ft., and the height to the eaves of the double-pitched frame is 13 ft.

The bottoms of the frames are fixed, and hinges are arranged so that the moments resisted by the ground beneath the bases are negligible. The use of this method of design would have made the double rafters difficult to transport; each of these members was therefore made in two pieces, which were connected at the site by two $1\frac{1}{8}$-in. post-tensioned Macalloy bars 15 ft. long.

The purlins were not prestressed; a constant L-shaped section was used so that about forty purlins could be made with each mould.

The architect was Mr. A. L. Heath. The frames and purlins were designed, made, and erected by Stuart's Granolithic Co., Ltd.

Fig. 1.

Fig. 2.

Rapid Construction with Sliding Shutters.

Two eleven-story structures, T-shaped on plan and providing 240 maisonette dwellings, have been built at Birkenhead with the use of sliding shutters. It is thought that this is the first time that sliding shutters have been used in this country in the erection of residential flats.

It was aimed to complete the work in twelve months, and it was decided to construct the floors together with the walls. The sequence of work was to raise the shutters for the walls continuously for a period of 24 hours, followed by a period of 24 hours for the construction of the floors, and so on for twenty days up to roof level. Provision for door and window openings was included in the sliding shutters. The electrical work, partitions, and

Fig. 1.

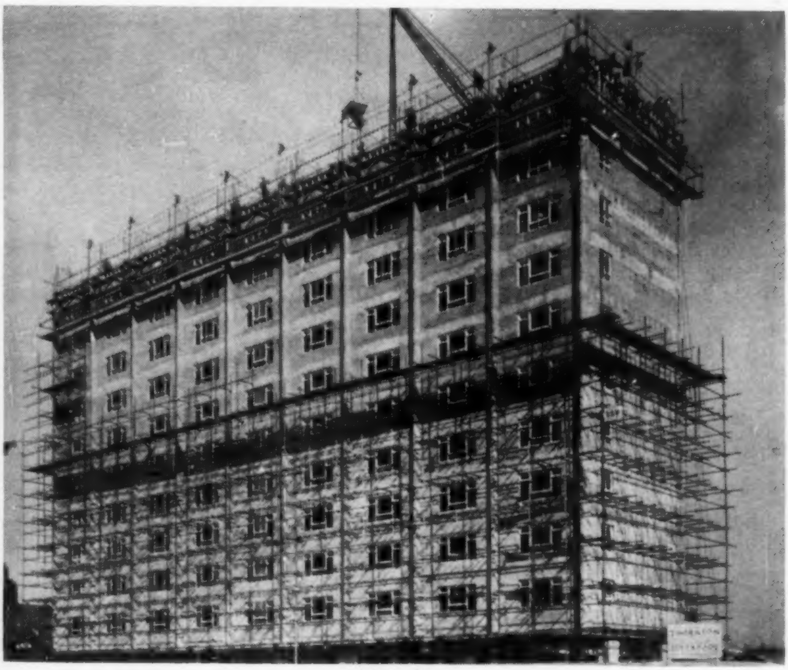


Fig. 2.

floor toppings were well in progress by the time the walls and floors were completed.

The structures are on piled foundations. The ground slabs (providing an area to be used for tenants' stores and laundries) were constructed first, and over these was constructed the lowest floor of the structure, supported on pairs of columns under each cross-wall. The sliding shutters were erected on this slab (*Fig.* 1). The cross-walls are 7 in. thick. Service ducts and refuse chutes were also constructed with the aid of the sliding shutters. *Figs.* 1 and 2 show the progress during eighteen days.

The "Prometo" system of sliding shutters was used, with the jacks spaced apart according to the load to be carried, and the shutters were raised at the rate of 1 in. in four minutes. The floors of the living-rooms comprise prestressed I-section beams with infilling blocks; timber floors are used for the bedrooms.

The main access stairs (*Fig.* 3) comprise precast flights and landings supported by L-shaped brick columns. These columns had cores of concrete, to the reinforcement of which the reinforcement projecting from the landings was tied to form a stable unit; the staircase was constructed at the same speed as the superstructure.

The concrete was mixed at a central plant and pumped to a hopper, from which it was delivered by crane to the working deck. A nominal 1 : 2 : 4 mixture was used, with ordinary Portland cement.

Piling commenced on November 19, 1956. The first 40 dwellings were completed on September 12, 1957, the next 60 dwellings on October 26, 1957, and the remainder in December 1957.

The architects were Mr. T. A. Brittain, F.R.I.B.A., Borough Architect of Birkenhead, in collaboration with Mr. Donald Bradshaw, F.R.I.B.A., consulting architect to the main contractors, Messrs. William Thornton & Sons, Ltd. Messrs. Ove Arup & Partners were the consulting engineers.

Fig. 3.

Lightweight Reinforced Concrete Walls.

Reinforced concrete made with vermiculite aggregate was used to form walls around a stair-well in an extension to a factory in Kent. The extension included the construction of an additional story; the load-bearing capacity of the existing structure was limited, and a high degree of fire protection was required around the new staircase. For these reasons, walls 19 ft. long, 8 ft. 3 in. high, and 9 in. thick were cast in place, using a 2 : 1 mixture of vermiculite and cement. The reinforcement consisted of ½-in. vertical bars at 34 ft. centres and horizontal bars at 2 ft. centres. The density of the concrete was 36 lb. per cubic foot.

A Silo in Essex.

THE silo shown in *Figs.* 1 to 5, built for British Plimber, Ltd., at Rainham, Essex, has a capacity of about 400 tons of wood chippings. It consists of a bin about 62 ft. square and 34 ft. high from which are suspended four inverted conical hoppers. Each hopper has a diameter of 30 ft. at the top tapering to 3 ft. 6 in. at the outlet; the height is 20 ft. The soffits of the beams which support the hoppers and the walls of the bin are 22 ft. above ground, and these in turn are supported by columns around the periphery of the bin and one central column.

Fig. 1.

Fig. 3.

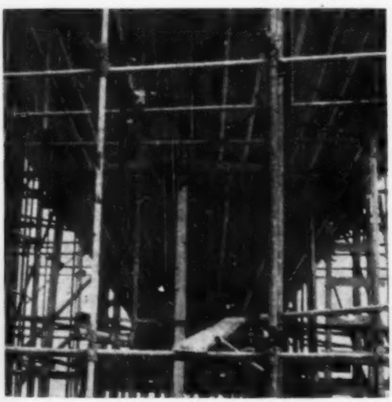


Fig. 2.

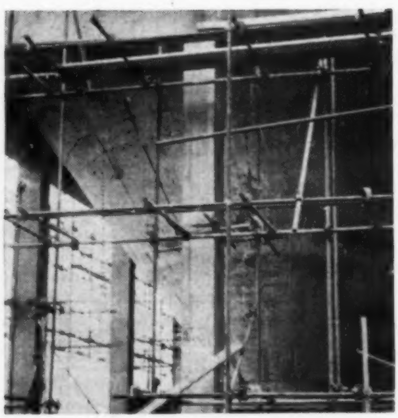


Fig. 4.

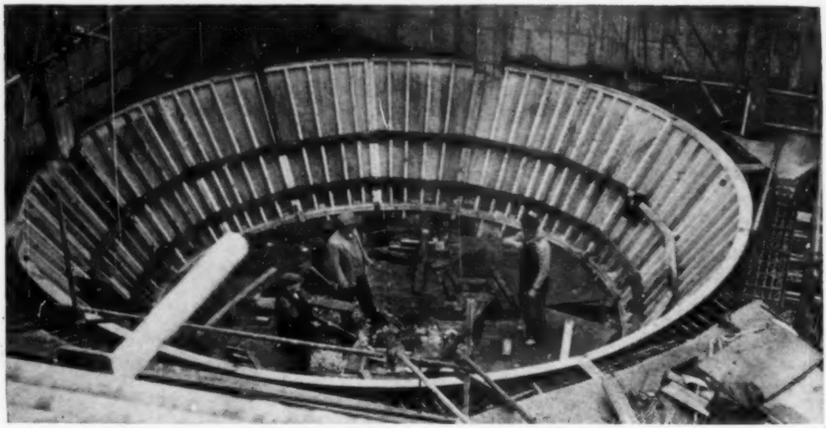


Fig. 5.

The floor of the bin has no horizontal surfaces.

The shuttering for the hoppers consisted of prefabricated panels with timber supports (*Figs.* 2 and 3).

The architects were Messrs. Douglas White & Furniss, the design was prepared by the Twisteel Co., Ltd., and the contractors were Messrs. F. Bradford & Co., Ltd.

A Road Bridge in Staffordshire.

THE level of the surface of the bridge (*Fig.* 1, page 45) over the River Churnet at the works at Froghall, Staffs, of Messrs. Thos. Bolton & Sons, Ltd., had to conform to existing road levels and the level of the underside of the bridge and the distance between the abutments were determined by the requirements of the River Board. It was considered desirable that the bridge should have no intermediate supports. The predetermined levels of the soffit and the road restricted the overall thickness of the deck slab to 1 ft. 1 in. The bridge comprises two precast main beams which also form the parapet, precast secondary beams at 4 ft. centres, and a deck slab cast in place.

The main beams, which are 51 ft. 8 in. long overall, were precast and prestressed by means of cables with twelve 0·276-in. wires, using the Gifford-Udall system. The concrete was specified to have a minimum crushing strength of 7000 lb. per square inch at 28 days. The beams were provided with lifting hooks at each end and were slung into position on prepared abutments with the aid of two cranes. The deck consists of reinforced concrete beams at 4 ft. centres. These are 12 ft. long and 7 in. deep, with a rebate on each side to receive precast concrete slabs which acted as permanent shuttering for the deck. The deck slab contains horizontal prestressing cables which enable the slab to span between the secondary beams and to combine with the main beams to form a prestressed trough. A 3-in. tarmacadam wearing surface is provided. The bridge is designed to carry the full Ministry of Transport loading.

The design was prepared by the British Reinforced Concrete Engineering Co., Ltd., who also supplied the reinforcement. The general contractors were Messrs. C. Cornes & Son, Ltd. The piling under the abutments was by the Cementation Co., Ltd.

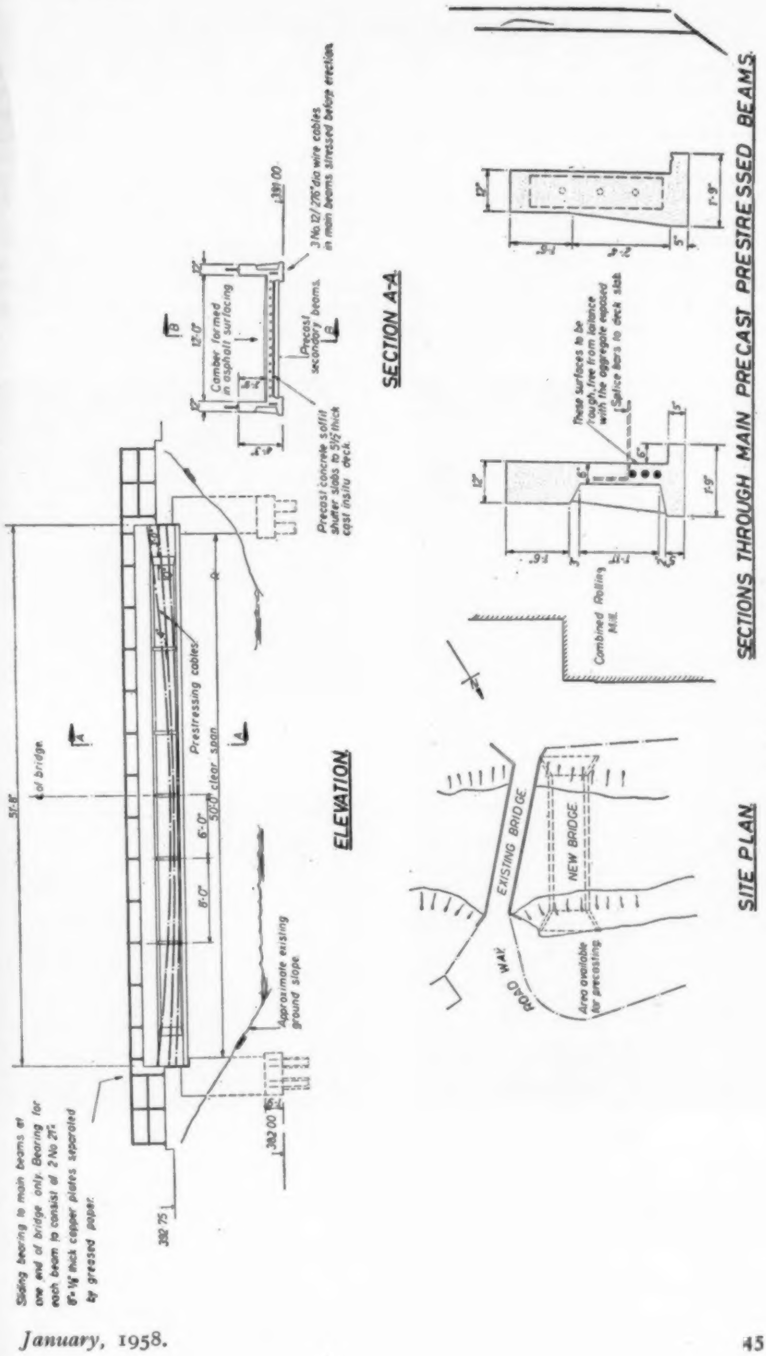


Fig. 1.—Road Bridge in Staffordshire. (*See page* 44.)

A Staircase at Colchester.

A STAIRCASE (*Figs.* 1 to 4) recently constructed for Messrs. Woods of Colchester, Ltd., provides access from the ground floor to the third floor. Each flight is semicircular in plan.

For æsthetic reasons it was desirable to avoid the use of vertical supports immediately underneath the flights, and an alternative design affording a lighter and more economical structure was chosen.

Each flight is supported by five cantilevers which radiate from one circular column of 17 in. diameter, which supports the complete staircase and is connected to the structural frame at each floor. The cantilevers merge into the thickness of the flight, providing a plain soffit throughout, and the exposed parts present a fanlike appearance when viewed from above (*Fig.* 1), which is in keeping with the firm's business as fan makers.

The architects were Messrs. Bailey & Walker, the reinforced concrete work was designed by the Indented Bar & Concrete Engineering Co., Ltd., and the contractors were Messrs. Henry Everett & Son.

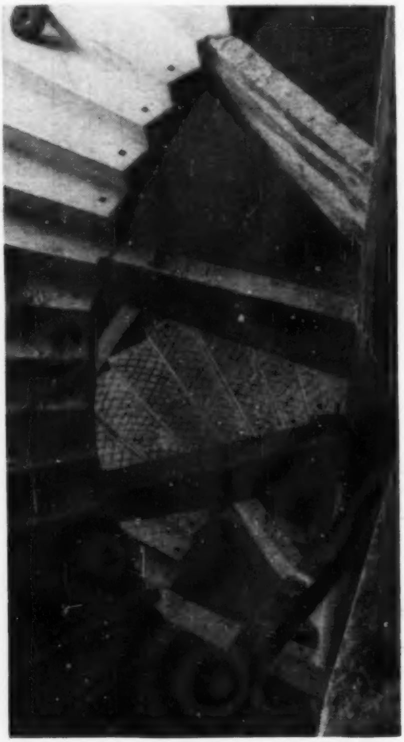


Fig. 1.

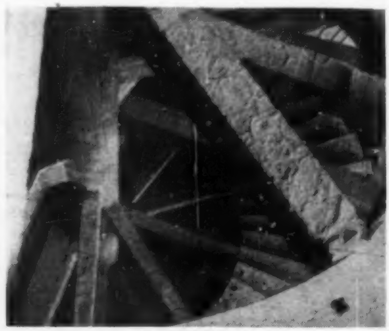


Fig. 2.

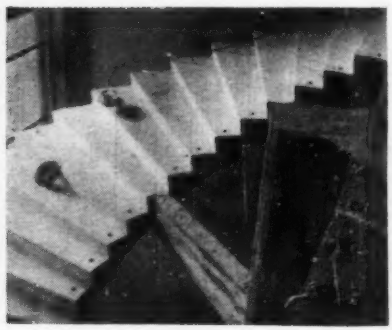


Fig. 3.

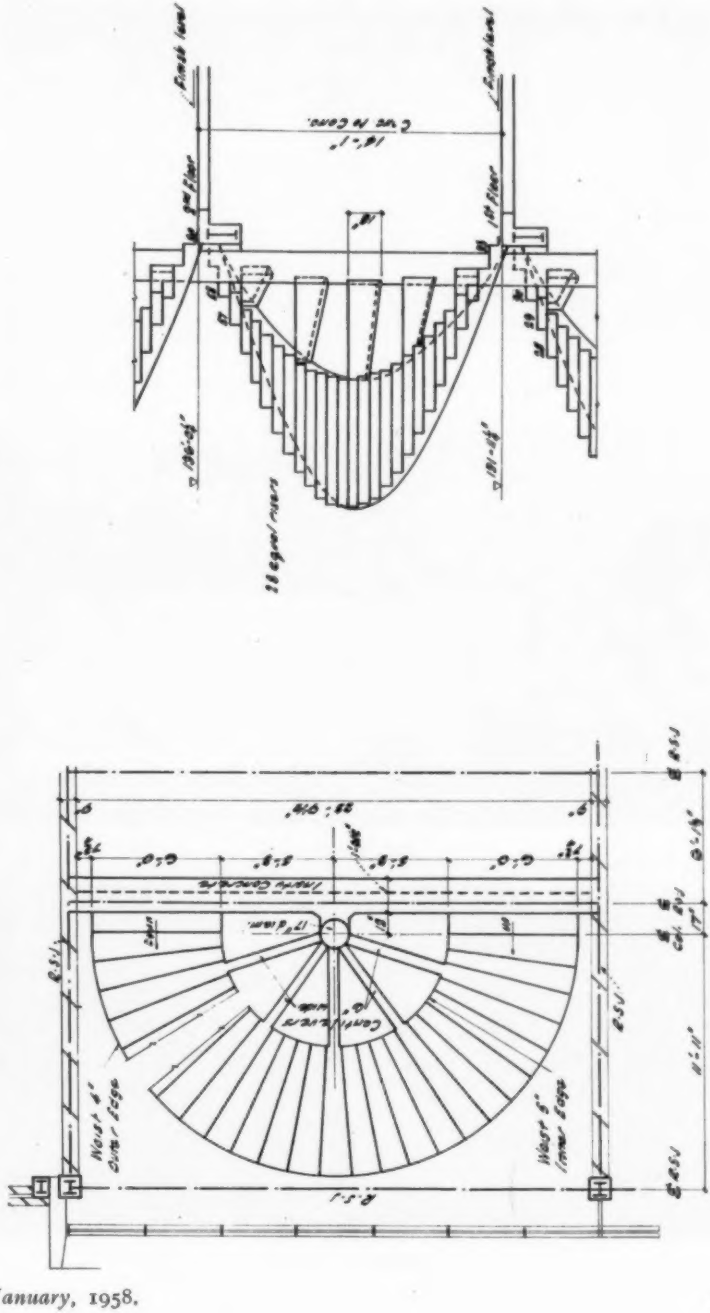

Finish level
2nd Floor
1st Floor
18 equal risers
Cantilevers
Waist 4" Outer Edge
Waist 5" Inner Edge
R.S.J.

Fig. 4.

Two Prestressed Railway Bridges.

INCLUDED in the improvements carried out by the National Coal Board at the Northumberland collieries of Lynemouth and Woodhorn are the extension and rearrangement of the sidings adjacent to the coal-preparation plant at Lynemouth and a new double-track railway connection between these sidings and the colliery at Woodhorn, where the tracks join the main line. The new connection is about 1½ miles long and passes over three roads. Of the three new bridges one is of structural steelwork on reinforced concrete piers and abutments. The other two comprise prestressed beams on reinforced concrete abutments. They are of similar design, but in one case the clear span is about 35 ft. and in the other about 55 ft. The shorter bridge (*Fig.* 1) consists of eighteen I-beams each 18 in. wide and 2 ft. 2 in. deep, prestressed with five Macalloy bars of 1⅛ in. diameter. The bars were passed through ducts formed in the concrete, a force of 32½ tons was applied to each, and colloidal grout was injected under pressure through holes in the end anchor-plates. After the erection of all the beams except the two side ones, two transverse bars were inserted and a force of 12 tons was applied to each (*Fig.* 2). The side beams were then erected and the bridge completed

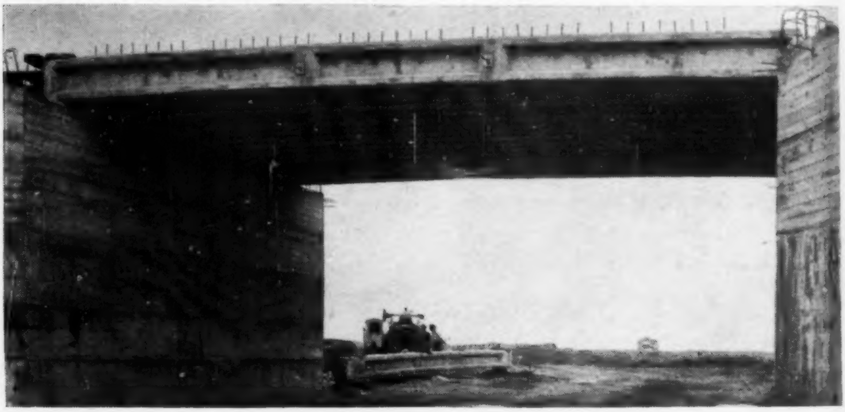


Fig. 1.

Fig. 2.

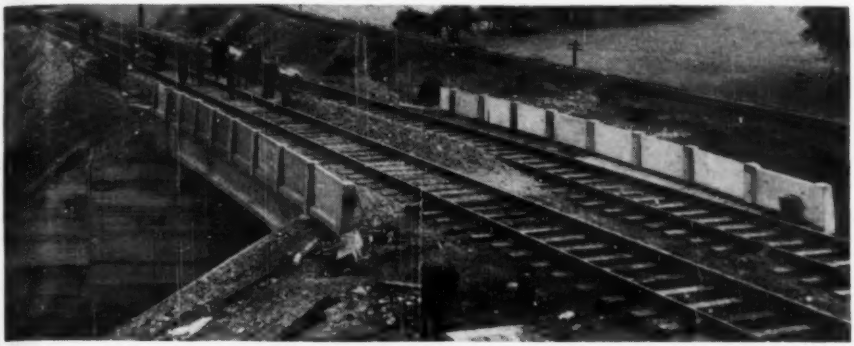


Fig. 3.

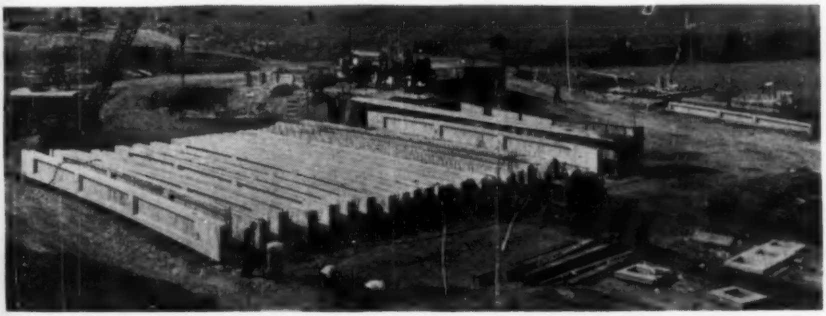


Fig. 4.

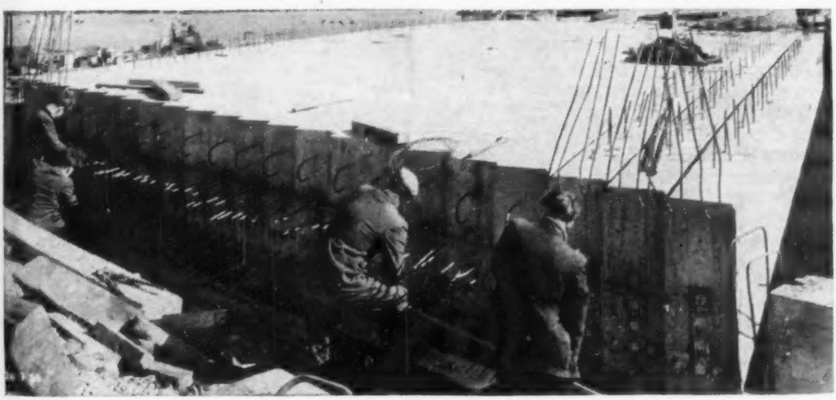


Fig. 5.

by the erection of precast concrete parapet panels and counterforts. The beams were cast with a camber of 1⅜ in. on the top surface and 2⅜ in. on the soffit.

In the case of the longer bridge (*Fig.* 3) there are twenty-two beams, each 56 ft. 8½ in. long, 15 in. wide, and between 3 ft. 6 in. and 3 ft. 9½ in. deep. They are prestressed by six Macalloy bars of 1⅛ in. diameter. The beams were cast with a camber of 2½ in. on the top surface and 6 in. on the soffit.

The minimum strength of the concrete was specified to be 7000 lb. per square inch at 28 days and 5000 lb. per square inch at the time when the prestressing force was applied to the concrete. A precasting yard (*Fig.* 4) was established near one of the bridges, and by means of steam curing a strength of 5000 lb. per square inch was obtained 16 hours after casting the beams. Sufficient bars were inserted and tensioned while the beams remained on the casting beds to enable them to be lifted; they were then placed in a storage area where the rest of the prestressing was done.

As the bridges were over public roads, which were closed while the beams were being erected, the period of possession was limited to 36 hours in the case of the longer bridge, and all the beams were transported, placed in position, and prestressed transversely within one period.

The abutments also act as retaining walls for the earth embankment of the railway, and provision was made for the level to be adjusted in the event of settlement. Bars projecting from the ends of the beams were cast into jacking beams (*Fig.* 5), and holes were left in the abutments for the insertion of hydraulic jacks (see *Fig.* 1).

The bridges cost about £22,000. The consulting engineers were Messrs. Posford, Pavry & Partners, and the main contractors were Messrs. Holloway Brothers (London), Ltd.

A North-light Roof with Warren Girders.

THE need for unrestricted working space in a workshop for the repair of contractor's plant precluded the use of internal columns, and a north-light roof spanning 90 ft. was adopted in order to provide an even distribution of light at working level (*Figs.* 1 and 2—see facing page). To move plant on the workshop floor a gantry crane travels on rails on each side wall.

Ordinary north-light profiles of such a large span would have required very heavy valley beams with much prestressing steel. To avoid this, a Warren girder was used in the plane of the glazing, the upstanding member in the valley of each shell acting as the bottom boom of the girder and the head of the adjacent shell being used as the top boom. This resulted in a very light valley construction.

Most of the concrete was cast in place, but the raking members of the Warren girders were precast with reinforcement projecting from their ends to connect with the shells and the valley beams.

The tensile forces in the end members of the Warren girders were fairly high, and it was decided to resist these by prestressing. Sufficient prestressing force to counteract normal tensile stresses in the gutter was provided by four Gifford-Udall cables each containing twelve 0·276 in. wires. By bending up one of these cables into each of the last two tensile members of the girder, the tension in the members was eliminated.

The shuttering for the shells incorporated resin-bonded plywood supported on a staging of tubular scaffolding.

The structure was built by Messrs. Peter Lind & Co., Ltd., for their own use. Messrs. Ove Arup & Partners were the consulting engineers.

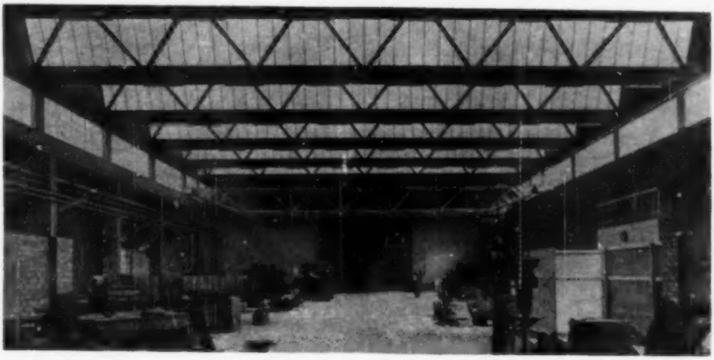


Fig. 1.

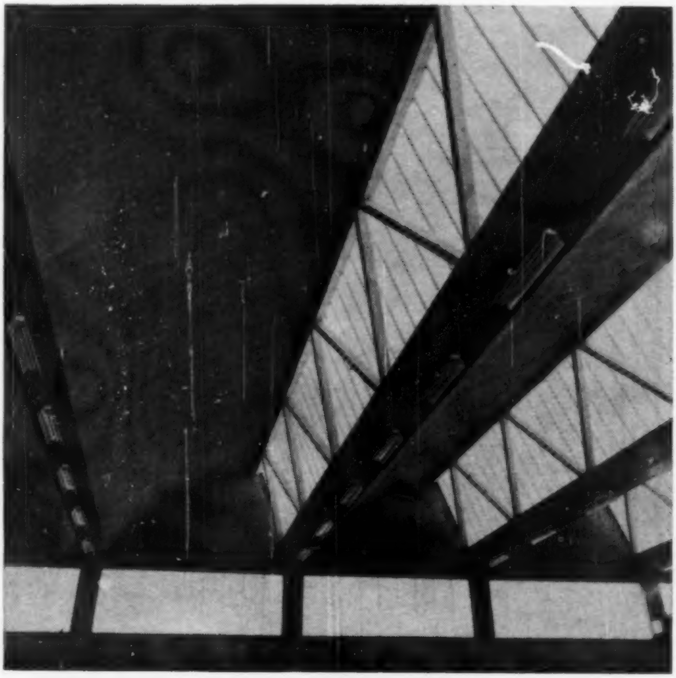


Fig. 2.

A North-light Roof with Warren Girders.

(*See facing page.*)

A Tall Building in Manchester.

An office building (*Fig.* 1) now being built for the Ministry of Works in Bridge Street, Manchester, is 160 ft. long, 48 ft. wide, and 195 ft. high. The frame is of reinforced concrete and the walls are mainly glass, the exposed edges of the floors and the faces of the external columns being covered with Portland stone. The ends of the building will be faced with Portland stone, which is being built at the same time as the concrete walls and acts as shuttering to one face of the walls. The four lower floors are of solid reinforced concrete and the 5th to 17th floors are hollow-tile slabs 10 in. thick.

The lift and staircase enclosures are at each end. The walls are 9 in. thick and are concreted in lifts of 9 ft., corresponding to the height of a story. Compaction is by poker vibrators. Ready-mixed concrete is used, the proportions for concrete with a strength of 4000 lb. per square inch at 28 days used in the hollow-tile floors being 560 lb. of cement, 1000 lb. of sand, 2340 lb. of ¾-in. limestone, and 340 lb. of water per cubic yard. In the case of the concrete with a strength of 6000 lb. per square inch at 28 days, as used for the frame, the proportions are 745 lb. of cement, 1850 lb. of ¾-in. gravel, 300 lb. of ⅜-in. gravel, 830 lb. of sand, and 320 lb.of water per cubic yard.

The consulting engineers are Messrs. R. Travers Morgan & Partners and the contractors Messrs. J. Gerrard & Sons, Ltd.

[Notes on the tower crane used in the construction of this building are given in this journal for December 1957.]

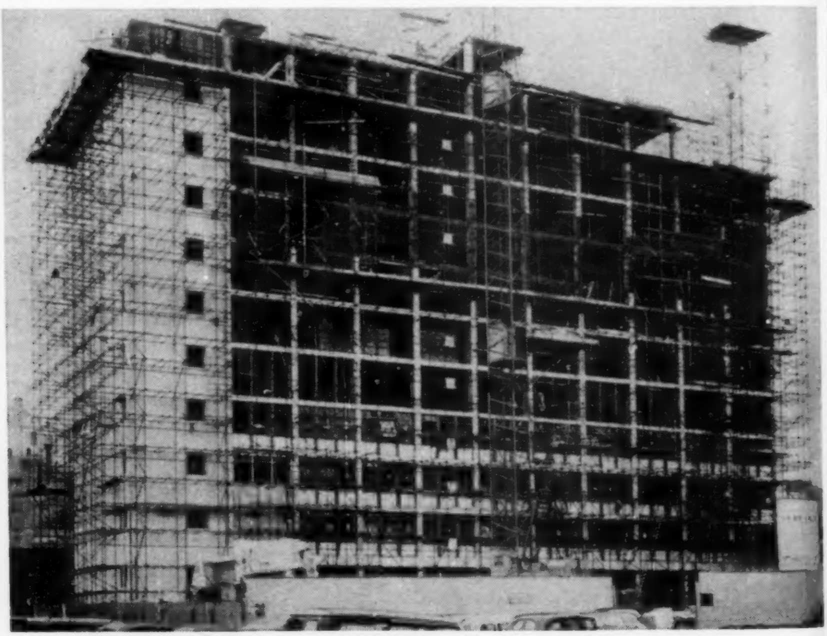


Fig. 1.

Replacing the Foundation of an Acid Plant.

By L. A. BARNES.

A DEFECTIVE foundation supporting an acid plant at a large chemical works has recently been demolished and replaced. The foundation was previously constructed in reinforced concrete, using high-alumina cement; its length was about 60 ft. and the cross section was that of a box, the dimensions of which were 10 ft. 6 in. deep by 9 ft. wide with walls and top and bottom slabs 18 in. thick. The top of the upper slab was at ground level.

The foundation originally carried three oleum and acid towers (*Fig.* 1), and during the erection of a fourth tower it was discovered that the interior of the foundation was being attacked by a highly acidic liquid. Inspection showed that the acid had seeped through the base and the lower part of the walls, which were built in 1951, and which were so severely attacked that in places they had disintegrated throughout their thickness and $\frac{7}{8}$-in. steel reinforcement bars had been completely corroded (*Fig.* 3). The liquid had a pH value of about 1·0, and remedial measures including the diversion of the liquid and the reconstruction of the foundation were necessary.

It was essential to avoid interruption of production during the reconstruction, and the presence of complicated pipe work (*Fig.* 4) with rigid joints made it necessary to carry out all constructional work from inside the existing box foundation.

The method of constructing the new foundation is shown in *Fig.* 2. As the upper slab of the old foundation was in good condition it was retained. The base slab and side walls were demolished in sections and replaced with concrete made with high-alumina cement and protected

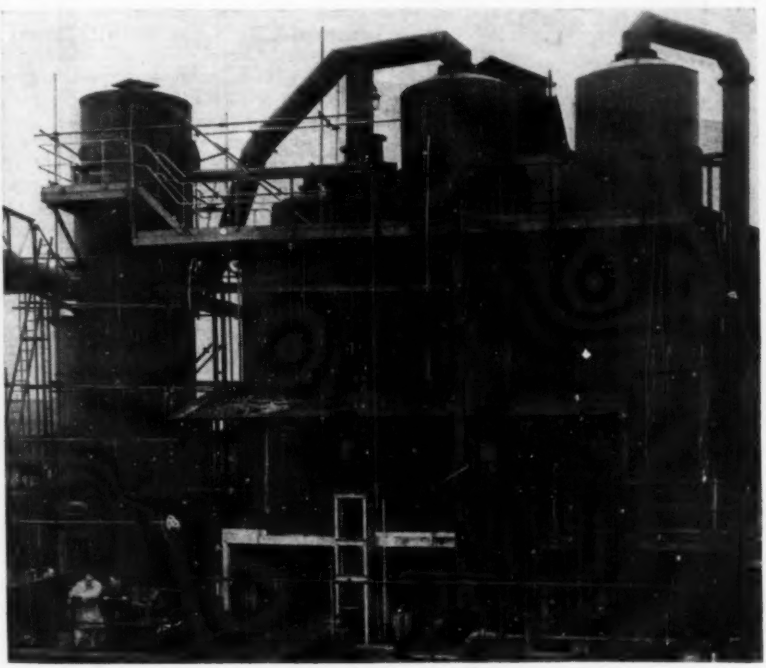


Fig. 1.

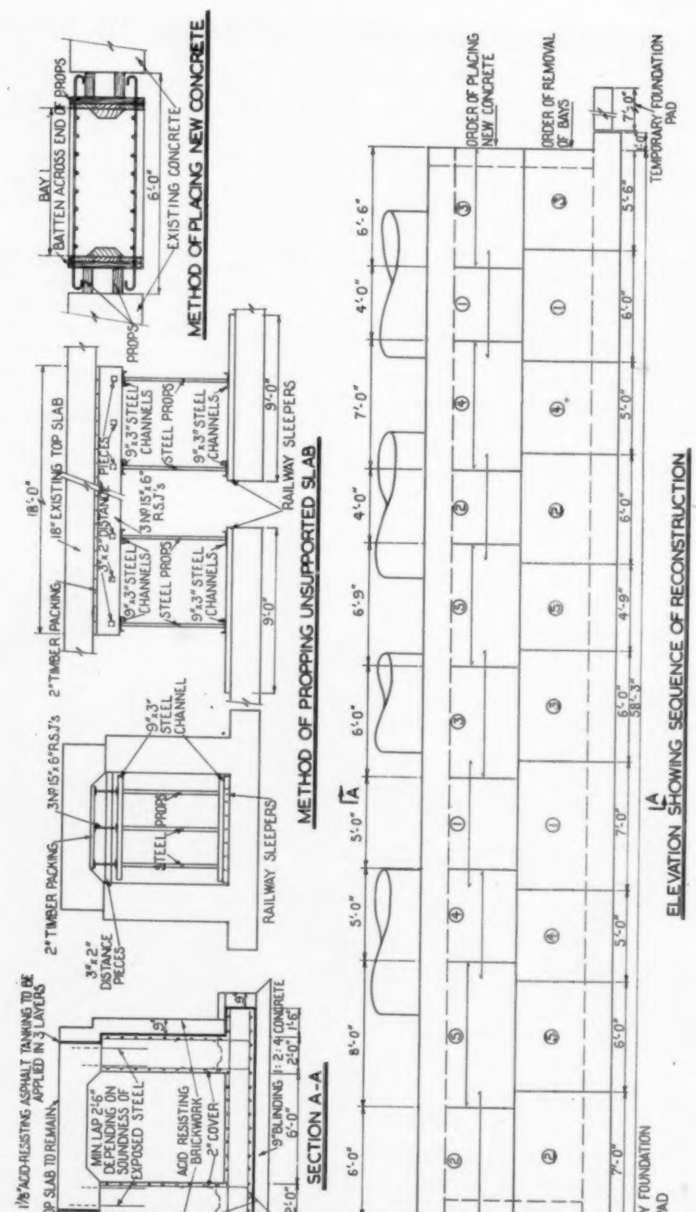

METHOD OF PLACING NEW CONCRETE
BAY 1
BATTEN ACROSS END OF PROPS
EXISTING CONCRETE
PROPS
6'-0"
METHOD OF PROPPING UNSUPPORTED SLAB
2" TIMBER PACKING
18" EXISTING TOP SLAB
18'-0"
3"x2" DISTANCE PIECES
3 No 15"x6" R.S.J.'s
9"x3" STEEL CHANNELS
STEEL PROPS
RAILWAY SLEEPERS
9'-0"
SECTION A-A
1/8" ACID-RESISTING ASPHALT TANKING TO BE APPLIED IN 3 LAYERS
EXISTING TOP SLAB TO REMAIN
MIN. LAP 2'-6" DEPENDING ON SOUNDNESS OF EXPOSED STEEL
ACID RESISTING BRICKWORK
2" COVER
3" COVER
9" BLINDING
1:2:4 CONCRETE
ELEVATION SHOWING SEQUENCE OF RECONSTRUCTION
ORDER OF PLACING NEW CONCRETE
ORDER OF REMOVAL OF BAYS
TEMPORARY FOUNDATION PAD
58'-3"

Fig. 2.

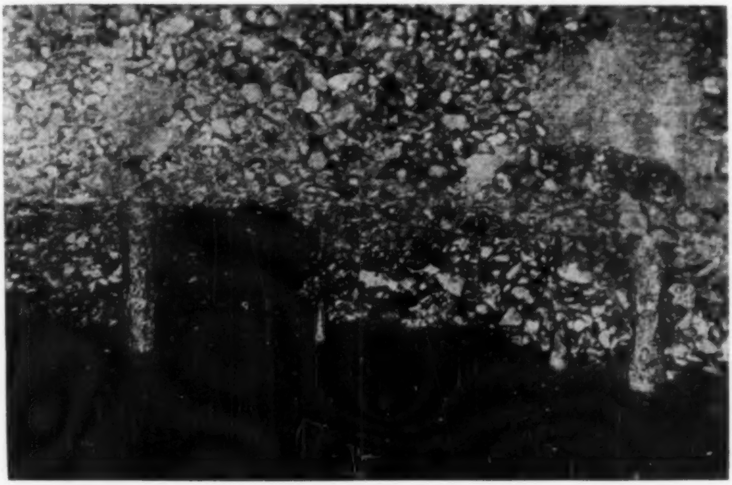


Fig. 3.

by an acid-resistant asphalt lining. Care was taken in the design and in the sequence of operations to avoid differential settlement during construction of the new work and permanent differential settlement on completion of the work. Flexible pipe connections were provided between the cooling pond and the towers, and temporary supports were provided for the towers during the period of reconstruction. Despite the difficult conditions under which the work was carried out, no settlement has yet occurred.

Messrs. George Wimpey & Co., Ltd., carried out the investigation, design, and construction of the work.

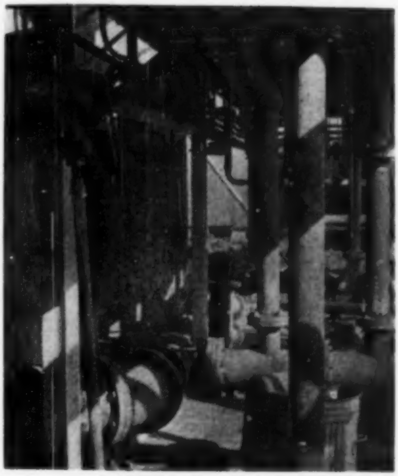


Fig. 4.

Electronic Computer for Designing Roads.

The development of a new method for the design of highways is now being completed at the Massachusetts Institute of Technology. By this method the information obtained by aerial photogrammetry is stored in an electronic computer. The direction and cross-sectional profile of the proposed road are then inserted into the computer, which calculates the geometry of the centre-line of the road, the cross sections, and the quantities of earthwork. An attempt is being made to produce a method that will also estimate the cost of the work.

Berkeley Nuclear Power Station.

THE construction of the reactors for the nuclear power station at Berkeley (*Figs.* 1 to 3), which was started in January, 1957, required the excavation of 250,000 cu. yd. of earth, the importation of large quantities of hard filling, the placing of 80,000 cu. yd. of high-strength concrete and 40,000 cu. yd. of lean concrete, and the erection of 9000 tons of reinforcement and 1400 tons of structural steel.

Eight months after work started a concrete base 30 ft. deep was ready for the pressure vessel in reactor No. 1, and the first lifts of concrete for the foundation of reactor No. 2 were completed. At this time most of the concrete had been placed in the foundation of reactor No. 1, on which work in the preceding months had been concentrated. The construction of reactor No. 2 is following a similar programme, and the work has proceeded simultaneously in the two excavations.

Each lift of concrete for the foundation of reactor No. 1 was divided into bays containing between 70 cu. yd. and 190 cu. yd. The concrete for the retaining walls and the foundation for the boilers, which will encircle the reactor, was placed at the same time. On October 20 the bottom dome of the pressure vessel (which had been constructed at the same time as the foundation) was moved into position. As the assembly of the vessel proceeds, the concrete to form the biological shield is being placed around it. This shield will be of concrete of high density; it will be 8 ft. 6 in. thick and placed in lifts 3 ft. deep.

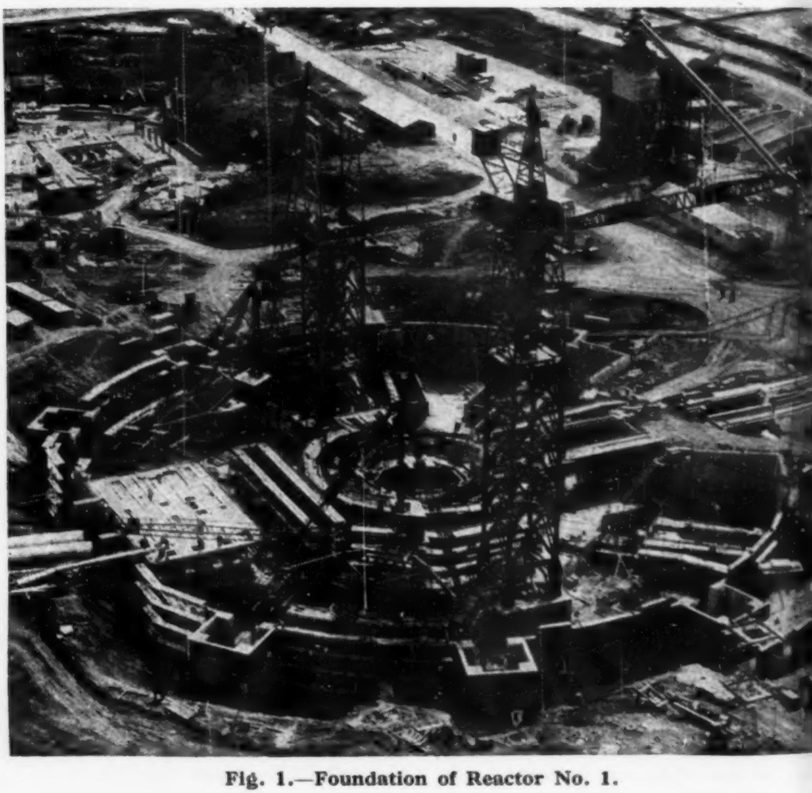


Fig. 1.—Foundation of Reactor No. 1.

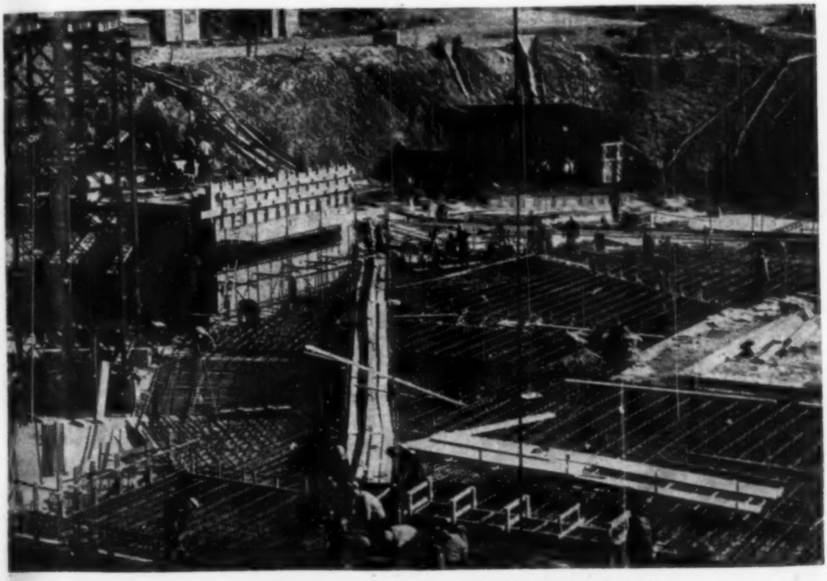


Fig. 2.—First Lift of Concrete for Reactor No. 1.

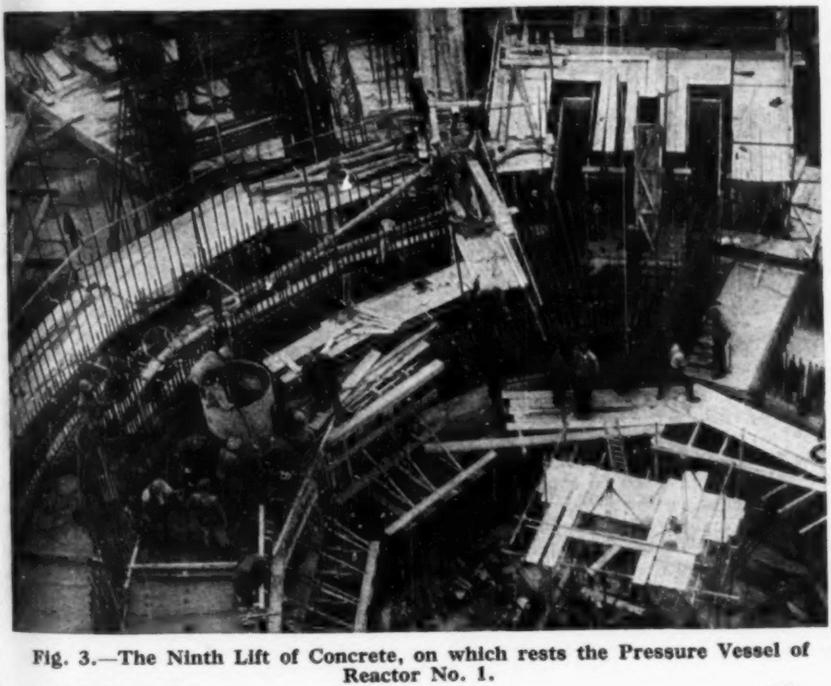


Fig. 3.—The Ninth Lift of Concrete, on which rests the Pressure Vessel of Reactor No. 1.

Fig. 4.—Arrangement of Concrete Mixing Plant.

A tower crane is in use on each side of reactor No. 1, and two more are being erected for the construction of reactor No. 2. Each crane is 130 ft. high, and is supported on a concrete base 15 ft. thick; the capacity is 15 tons at a radius of 90 ft. Between the two reactors excavation of the cooling pond is completed and concreting has begun.

A large batching and mixing plant (*Fig.* 4) started working in May. The tower is 90 ft. high, and the plant can produce 700 cu. yd. of concrete a day. Five types of aggregate are brought by lorry and tipped into storage bins at ground level with a capacity of 4000 cu. yd. Each bay of the storage area has hopper-gates operated by remote control from a penthouse at the top of the plant. The required quantities of aggregates fall through the gates on to a conveyor-belt in a tunnel below the storage bins and are carried to an octagonal hopper at the top of the tower, whence they are discharged into weighing hoppers 41 ft. above ground. Loose cement is unloaded into a hopper at ground level and lifted into four silos, each of which has a capacity of 125 tons, by air-slides and a bucket-elevator.

From the weighing hoppers the materials are delivered to two mixers, each of 2 cu. yd. capacity, 14 ft. below the hoppers. The mixed concrete is discharged into two hoppers, one of which supplies lorries underneath it and the other supplies a reversible conveyor which transports the concrete to four pumps. Each pump is driven by a 55-h.p. motor and has a capacity of 15 cu. yd. per hour. The concrete is pumped through 6-in. pipes to the working areas, where it is compacted by pneumatic poker vibrators. The final lifts of concrete will be pumped from ground-level to a height of over 100 ft. Steam-heating plant is installed beside the batching plant for use in case of frost.

The consulting engineers are Messrs. W. S. Atkins & Partners, and the general contractors are the A.E.I.–John Thompson Nuclear Energy Co., Ltd. Messrs. John Laing & Son, Ltd., are the civil engineering contractors for the work described, and Messrs. Balfour, Beatty & Co., Ltd., are also working on the site.

Airport Terminal Buildings.

THE new terminal building at Gatwick Airport (*Fig.* 1) will be accessible by road and rail. The main road passes under the building, to which it is connected by a reinforced concrete bridge and an elevated roundabout, and the adjacent railway station is connected to the building by a bridge of 70 ft. span. Passengers will pass through the building at the level of the concourse and on to the first floor of the " pier ", which will be a two-story building 21 ft. wide and about 900 ft. long. Access to aircraft adjacent to the pier will be by stairs at intervals along the building.

Fig. 1.

The terminal building will be 350 ft. long and 130 ft. wide and will be of reinforced and prestressed concrete. It will be generally a three-story structure, with a mezzanine floor between the ground-floor and first floor at the end nearest the aprons. The first floor, or concourse, will consist of prestressed beams extending almost the whole length of the building. The beams will be at 20 ft. centres, and top and bottom slabs will form large ducts for baggage conveyors. The prestressed beams span about 70 ft. over the main road, 60 ft. over the perimetral road which also passes under the building, and 60 ft. over a new telephone exchange. The columns carrying this floor are necessarily large; their foundations are reinforced concrete rectangular bases on clay with a permissible pressure of 3 tons per square foot. The beams which support the remainder of the building will be of composite I section and will comprise precast webs, which are prestressed to carry the weight of the beam and slabs, and concrete cast in place to form the floor slabs. Ducts are formed in the concrete to accommodate prestressing cables, the Freyssinet system being used throughout.

The remainder of the structure will be of reinforced concrete using concrete of medium grade, high-tensile steel, and hollow-pot slabs. A central expansion joint will be provided.

Generally the surface finish of the columns and beams will be that produced by the sawn-board shutters. The walls of the building will be of steel and glass, suspended from a slab cantilevering 4 ft. 6 in. past the first internal row of columns. One wall is designed to be dismantled and

re-erected when the building is extended. Two rows of columns are designed to support a five-story reinforced concrete office building to be built later on top of the building. The consulting engineers are Messrs. Frederick S. Snow & Partners and the contractors Messrs. George Wimpey & Co., Ltd.

The new terminal building and apron at Amman Airport was constructed for the Hashemite Kingdom of Jordan by Messrs. Tabba'a & Nemeh Company. The work was completed in July 1956 at a cost of £129,000.

The building is designed to conform with the normal methods of construction of this type of building in Jordan, with a facing of local stone ashlar backed with weak concrete. The concrete was also graded in accordance with local practice, the specified crushing strength at 28 days for each of the four grades used being 3400, 2700, 2200, and 1700 lb. per square inch.

Rock was generally found within 1 ft. 6 in. of the ground surface. Separate bases were used for the columns and strip footings for the walls. The ground-floor slab is of plain concrete 12 in. thick laid directly on compacted stone filling. Ducts were left in the floor to accommodate the services. The floor surface generally consists of terrazzo files.

The suspended floors are of reinforced concrete 5 in. thick covered with thermoplastic tiles. The roof is of similar construction and is covered with asphalt, which is protected from solar radiation by a layer of crushed limestone chippings 2 in. thick.

The frame of the building is so designed that the columns are completely contained within the walls, the concrete of which is 8 in. thick at the bottom and 6 in. elsewhere. The walls are reinforced with $\frac{3}{16}$-in. bars at 12 in. centres each way.

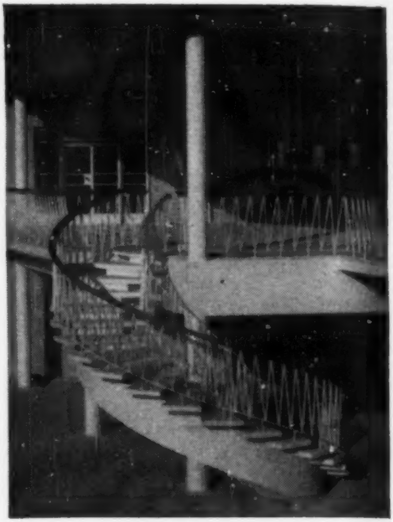


Fig. 2.

A helical staircase (*Fig.* 2), carried by a single beam, provides access to part of the first floor. The staircase turns through 270 deg. and the reinforced concrete treads were fastened to the beam by passing the reinforcement of the treads through the beam; the ends of the reinforcement are threaded and the treads secured by nuts.

The whole of the construction was done by local labour. The coarse aggregate used was crushed local limestone, the sand was river bed sand, and the cement was imported from Syria.

The consulting engineers are Messrs. Frederick S. Snow & Partners.

Congress on Prestressed Concrete.

THE third International Congress of the Fédération Internationale de la Précontrainte is to be held in Berlin from May 5 to May 10. Full details may be had from the organising secretary, Dipl.-Ing. P. Misch, at Deutscher Beton-Verein, 61 Bahnhofstrasse, Wiesbaden, P.O. Box 543, Germany.

The Reinforced Concrete Association.

A PROVISIONAL committee of the Reinforced Concrete Association has been appointed to start a branch in the West of England. The inaugural meeting will be held early this year. Details may be obtained from the Secretary of the Association, at 94–98 Petty France, London, S.W.1.

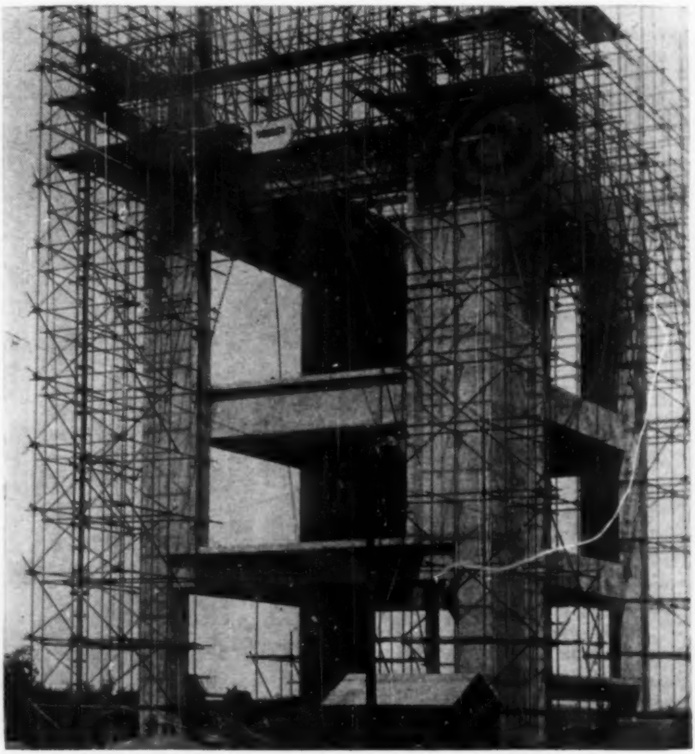
A Colliery Winder Tower.

A REINFORCED concrete winder tower has been constructed at Weetslade Colliery for a shaft which has yet to be sunk. The tower will be used for sinking the shaft before being completed. It is about 42 ft. by 45 ft. in plan and about 105 ft. high from ground level to the top of the penthouse roof, the main roof level being 11 ft. below this.

The winder floor is 65 ft. 4 in. above ground level and accommodates a multiple-rope friction winder and an auxiliary winch with a travelling crane for purposes of erection and maintenance. This floor is designed to resist a breaking load in the ropes of 220 tons, acting at the centre of the floor. About 24 ft. 6 in. below is a floor which carries the switchgear, motor-generator set, standby compressor, and other plant, and is provided with a mono-rail suspended beneath the winder floor for servicing the plant. The penthouse accommodates a sheave to enable the ropes to be changed by the auxiliary winch; it also houses filters which allow clean air to pass over the winder and auxiliary plant and thence down the shaft as part of the ventilation of the mine. A 10-cwt. lift will serve the ground floor and the auxiliary and winder floors, and a reinforced concrete stairway provides access to all floors.

The structure consists of reinforced concrete columns at each of the four corners, with beam-and-slab floors at each level providing clear spaces for plant. The tower stands upon a collar which contains the first 30 ft. of the shaft and which is supported on piles.

During its present period of use as a headgear for shaft-sinking, temporary structures are erected upon the winder



floor and auxiliaries floor to support the sheaves for the scaffolding, kibble, etc.; the ropes are carried to a temporary winch and winder house at ground level.

The consulting engineers are Messrs Posford, Pavry & Partners, and the contractors are Messrs. George Wimpey & Co Ltd., and Messrs. J. L. Kier & Co., Ltd.

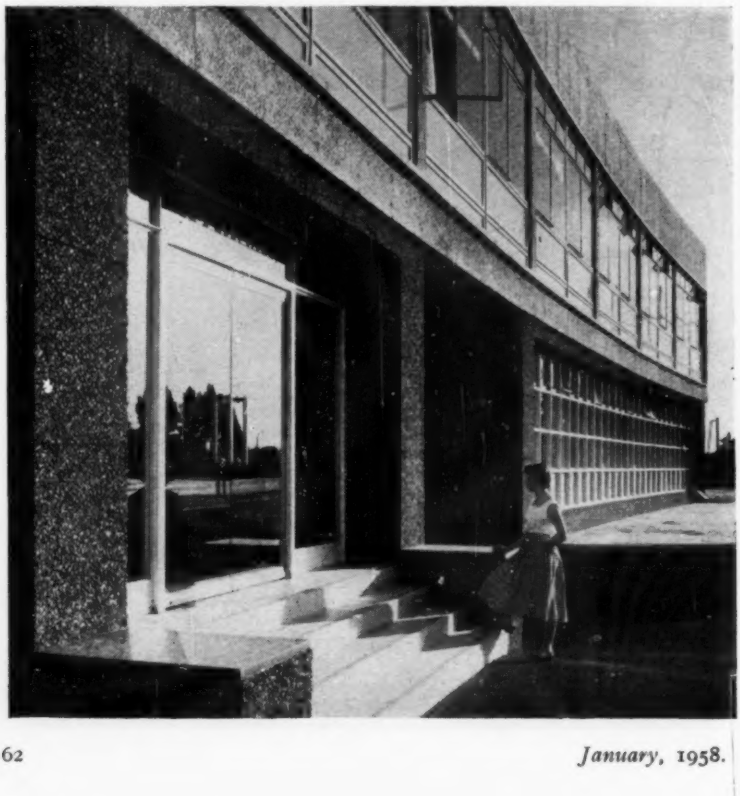
Precast Wall Slabs.

A FEATURE of a building recently completed on the Great West Road, London, for Gevaert, Ltd., is precast slabs used for the outer walls. The structure comprises reinforced concrete columns and beams cast in place and hollow-tile floors. The columns are 9 in. wide and are within the thickness of the walls.

The front and end walls are of brick with precast facing slabs, $1\frac{1}{2}$ in. thick, with an exposed-aggregate finish. There is a 1-in. cavity between the slabs and the inner leaf of brickwork. The slabs for the lower story are made with green Genoa marble aggregate and black Portland cement, and the aggregate is exposed; those for the upper stories are made with white marble and white cement and are lightly polished. The slabs are fixed by means of gunmetal clamps to the concrete frame or to concrete blocks set in the backing bricks.

The architects are Messrs. Douglas & J. D. Wood in association with M. Georges Lust. Mr. J. Bak is the consulting engineer.

Messrs. Gee, Walker & Slater, Ltd., were the main contractors, and the Malacarp Terrazzo Co., Ltd., were responsible for the precast slabs.



A Reservoir at West Hartlepool.

FLAT-SLAB ROOF.

THE reservoir shown in *Figs.* 1 and 2, recently constructed for the Hartlepools Water Company, is to be used for domestic supply only. It is supplied by a single main of 18 in. diameter from an electrically-driven submersible pump situated about 100 ft. from the reservoir.

The outside of the reservoir measures 100 ft. by 101 ft. It is divided by a central wall into two compartments each having a floor area of 48 ft. 1½ in. by 96 ft. 10 in. The average height from the floor to the underside of the roof is 18 ft. 9 in. and the maximum depth of water is 17 ft. 3 in.

All the walls vary uniformly in thickness from 9 in. at the roof to 19 in. at the bottom. The bases of the outer walls are 13 ft. 5 in. wide and their thickness varies from 2 ft. at the outer face of the walls to 1 ft. 1½ in. The base of the partition wall is 26 ft. 2 in. wide and varies in thickness from 1 ft. 9 in. to 1 ft. 1½ in. The floor is 6 in. thick generally and is laid on a 3-in. blinding layer. The 6-in. slab is separated from the bases of the walls by watertight joints, and the edge of the slab rests on a ledge 6 in. square formed in the tops of the bases. The joint between the slab and the bases allows for rotation and lateral movement, and is formed with a jointing material ⅜-in. thick vertically and building paper horizontally. Watertightness is ensured by a rubber waterstop. In each compartment the floor is divided into two panels by a contraction joint formed with building paper and a plastic waterstop; each panel measures 25 ft. by 37 ft.

Fig. 3 shows the roof of the reservoir, which is of flat-slab construction with "dropped" panels, the slab being 7½ in. thick and the panels 10½ in. thick. It is supported by columns, 12 in. square, with enlarged heads. In each compartment there are three rows of six columns, of which only four columns of the central row have their bases built monolithically with the floor slab. The other columns are built on the wall bases. The columns are spaced at centres of 15 ft. in one direction and 15 ft. 10 in. in the other.

The valve-control house on top of the reservoir has reinforced concrete walls 6 in. thick and a roof 4½ in. thick. Each

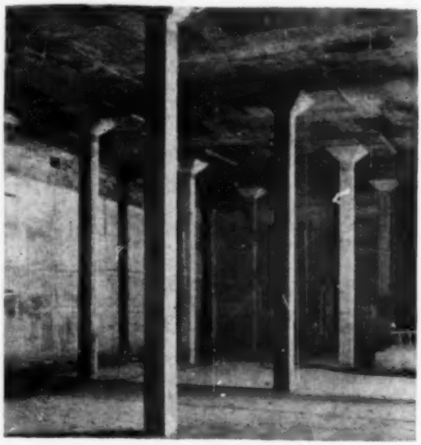


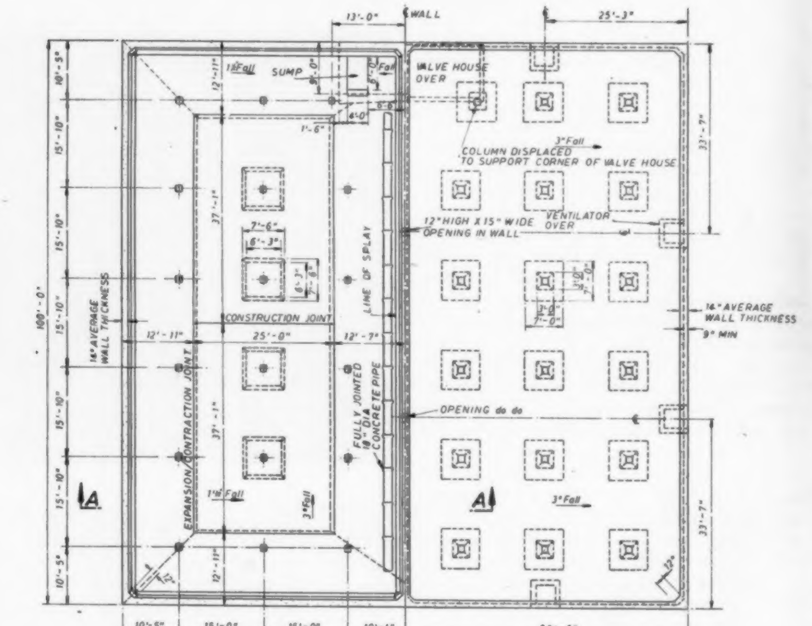
Fig. 1.

compartment of the reservoir can be entered from the control house through 3-ft. square manholes. The control house also contains depth-recording apparatus, the float recorders working in concrete tubes of 15 in. diameter. The water level in the reservoir is controlled automatically by a device which actuates a relay mechanism in the pump house. A lightning conductor is provided.

At the bottom of the walls porous concrete pipes of 4-in. diameter are laid between manholes at each corner of the reservoir to carry away rainwater and water that may leak from the reservoir. At one of the manholes the water enters a wash-out pipe which carries it to a nearby stream. All the manholes are of precast concrete surrounded by plain concrete 8 in. thick.

The walls of the reservoir are embanked with earth at a slope of 1½ to 1, and the roof is covered with 9 in. of soil on a layer of coarse gravel 3 in. thick.

The ground on which the reservoir is built consists of glacial drift, mostly boulder clay with some sandy clay, about 80 ft. deep. Tests showed that the permissible bearing pressure about 5 ft. below ground level was 1·25 tons per square foot. Ground-water was encountered at 7 ft. 6 in. The level of the reservoir floor



HALF PLAN SHOWING FLOOR
HALF PLAN SHOWING ROOF OVER
SECTION A - A.
SUMP
VALVE HOUSE OVER
COLUMN DISPLACED TO SUPPORT CORNER OF VALVE HOUSE
12" HIGH X 15" WIDE OPENING IN WALL
VENTILATOR OVER
CONSTRUCTION JOINT
EXPANSION/CONTRACTION JOINT
LINE OF SPLAY
FULLY JOINTED 18" DIA CONCRETE PIPE
14" AVERAGE WALL THICKNESS
9" MIN
OPENING do do
100'-0"
12" EARTH COVER (3" GRAVEL, 9" SOIL)
1:1.5 SLOPE
TOP WATER LEVEL
7½" SLAB
6" SLAB
12"x12" COL
18" DIA. FULLY JOINTED CONC. PIPE
CONCRETE SUPPORT
3" BLINDING
RESERVOIR SYMMETRICAL ABOUT CENTRE-LINE

Fig. 2.

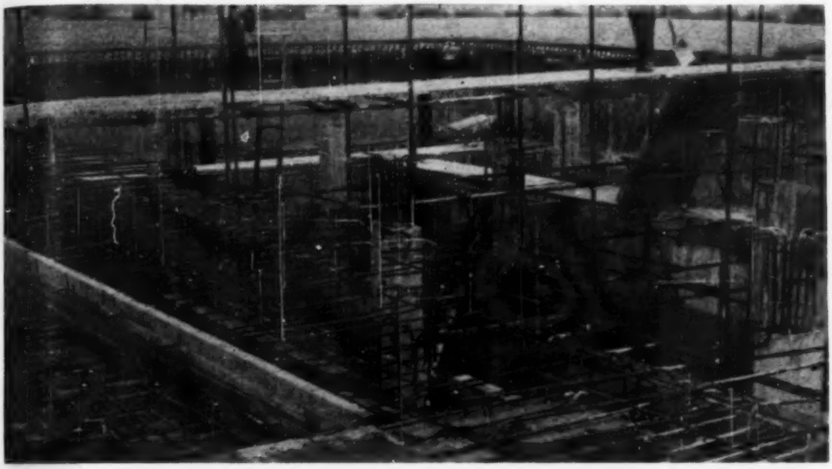


Fig. 3.

was determined by the level of the existing main, and consequently it was not possible to balance the excavation and the filling; surplus excavation was dumped near by, covered with top soil, and seeded.

The design of the reservoir is in accordance with the Code of Practice for Water-containing Structures. The stresses in the roof were limited to those permissible at a water-retaining face. In designing the bases of the walls the effect of column loading was taken into account; it was thus possible to omit an external projection which would otherwise have been necessary if the permissible ground pressure was not to be exceeded. The roof was designed for a load of 30 lb. per square foot in addition to the weight of the earth.

The specified minimum crushing strength of the concrete was 3750 lb. per square inch at 28 days. The average strength of the cubes tested was 4910 lb. per square inch and the minimum 3550 lb. per square inch. Fifty-eight cubes were tested, of which two were slightly below the specified strength.

The coarse aggregate was gravel graded from $\frac{3}{4}$ in. to $\frac{3}{16}$ in. The constituents of each batch of concrete were 224 lb. of ordinary Portland cement, 385 lb. of sand, and 717 lb. of coarse aggregate. The concrete was transported by a mobile crane in skips of $\frac{1}{2}$ cu. yd. capacity. Consolidation was by internal vibration.

The shuttering for the walls was intended to be 8-ft. by 4-ft. panels of resin-bonded plywood $\frac{3}{4}$ in. thick on timber frames, but because of the shortage of labour this method was used only for the dividing wall; the external shuttering was rough brickwork $4\frac{1}{2}$ in. thick. The concrete was placed in lifts of 4 ft., corresponding to the height of the shutter-panels. The shuttering for the roof consisted of steel pans supported by steel scaffolding. The columns and flared column-heads were shuttered with timber. The roof was concreted in panels, with construction joints midway between the columns in both directions.

When the reservoir was filled with water for testing purposes the permissible fall in level was $\frac{1}{16}$ in. in the second seven days, and this result was achieved. During the first week the fall was $\frac{7}{32}$ in. in one half of the reservoir and $\frac{3}{16}$ in. in the other.

The cost, excluding all pipework and similar fittings, but including site preparation, excavation, drainage, construction, backfilling, covering, soiling, seeding, and reinstatement was about £25,900. This corresponds to an overall rate of £22 per cubic yard of concrete.

Mr. James Clough is the Engineer and General Manager of the Hartlepools Water Company, the British Reinforced Concrete Engineering Co., Ltd., were the engineers, and the contractors were Dowsett Engineering Construction, Ltd.

A Factory at Southampton.

A SINGLE-STORY factory (*Fig.* 1) and administration building (*Fig.* 2) for the Western Manufacturing Co. (Reading), Ltd., were recently completed at Southampton. The factory comprises five bays (*Fig.* 3), each 50 ft. wide and 210 ft. long, providing a total floor area of 50,750 sq. ft. It has a precast frame. The columns, which are at 30 ft. centres, were placed in sockets formed in the foundation, and wedged and strutted in position; after the erection of the valley-beams the columns were aligned, levelled, and plumbed, and then grouted into the sockets. The valley-beams consist of balanced cantilevers of precast reinforced concrete (bolted to the heads of the columns) and central connecting members. The joints are stepped to provide the seating. The ends of the valley-beams and purlins are supported on brick gable walls. The eaves are 19 ft. 3 in. above ground level. The building has brick cavity walls and an asbestos-cement roof lined on the underside with insulation board.

The roof trusses consist of two precast reinforced concrete rafters, with steel ties and rods to prevent sagging; each truss was assembled in a vertical position on the ground, lifted by two mobile cranes, and lowered on to supports on the valley beams. The trusses are at 15 ft. centres, and connected by precast reinforced purlins with grouted joints.

The administration building has two stories; its framework is of reinforced concrete with precast columns. The first floor and the roof are of hollow-pot construction, and the ground floor is a reinforced concrete slab 6 in. thick. The front elevation consists of a combination of red and buff brick panels and concrete slabs with exposed-aggregate finish. The main entrance has a glass and hardwood screen and a cantilevered canopy of reinforced concrete. The roof is covered with vermiculite 2 in. thick, on which

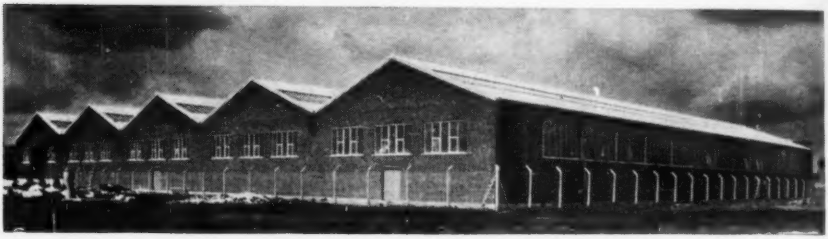


Fig. 1.

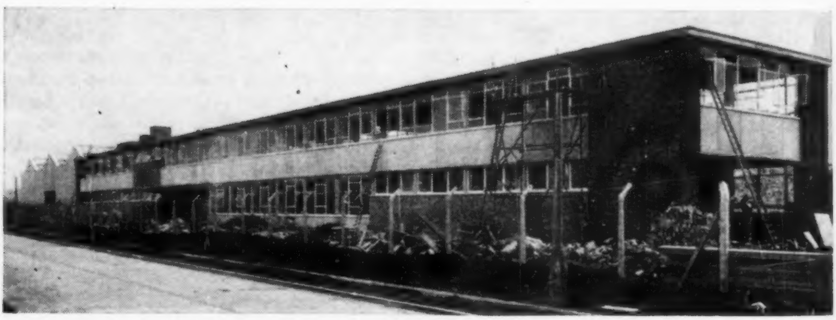


Fig. 2.

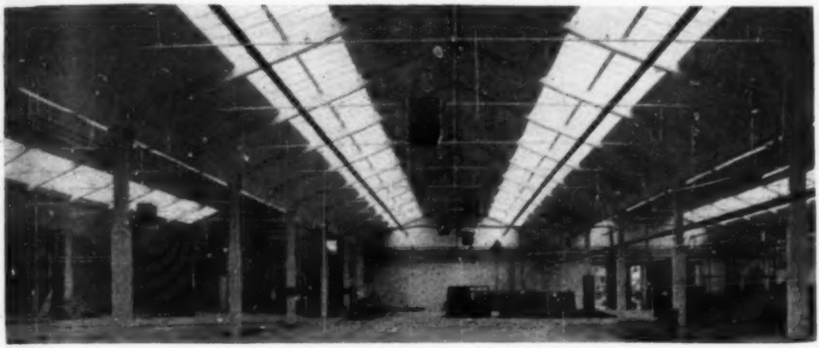


Fig. 3.

layers of asphalt were laid in which chippings of white spar are embedded.

The buildings were commenced on March 11, 1957, the factory was completed in August 1957, and the remainder of the work in October, 1957. The consulting engineers and architects were Messrs. C. W. Glover & Partners and the contractors Messrs. George Wimpey & Co., Ltd.

Precast Building at Wexham Springs, Bucks.

A TWO-STORY building (*Fig.* 2) about 170 ft. long and 50 ft. wide has recently been completed at the research department of the Cement and Concrete Association at Wexham Springs. The building is to be used for training courses, and includes a lecture room, lounge, recreation room, hostel, kitchen and canteen, and a printing department.

The structure comprises single-story precast wall units, precast main and secondary beams, precast floor slabs with structural topping cast in place, and interior columns cast in place. The wall units, in which panels with an exposed-aggregate finish are incorporated, were placed side by side to form the two main elevations. Cavities were formed between adjoining units (*Fig.* 1) in which reinforcement and concrete were placed; the infilling concrete, together with the sides of the units, form the outer columns. Two rows of internal columns, at 12 ft. centres, support precast longitudinal main beams. The precast transverse beams support precast slabs 2 in. thick covered with 2 in. of concrete cast in place.

In the kitchen and canteen there is a central line of columns, which, with the longitudinal beams which they support, were all cast in place. The floor consists of prestressed precast hollow beams.

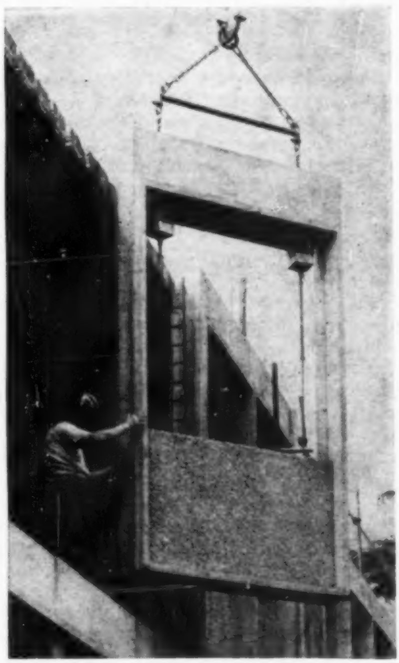


Fig. 1.

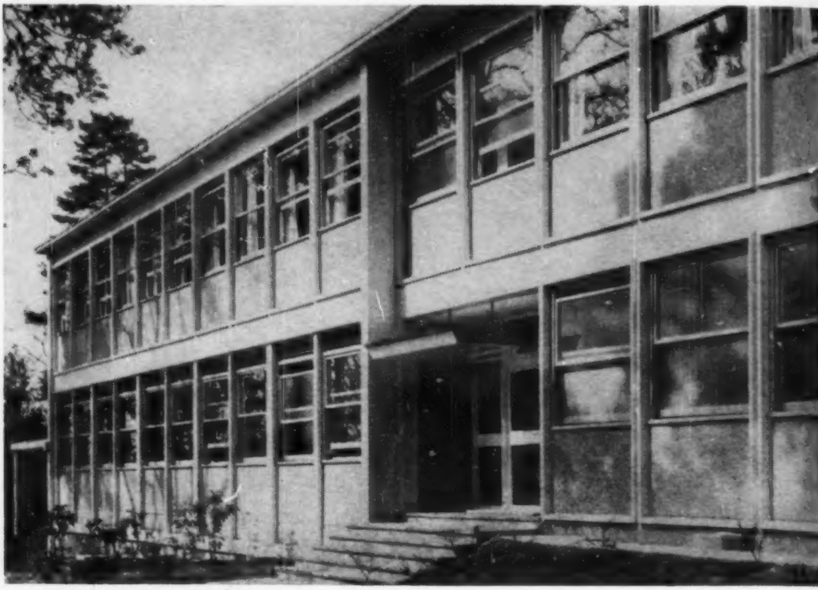


Fig. 2.

The end walls are of cavity construction, the inner leaf consisting of lightweight concrete blocks and the outer leaf of concrete slabs with an exposed-aggregate finish. The heating and oil-storage chamber in the basement has a reinforced concrete floor and walls 12 in. thick. The architect was Mr. W. R. Oram and the contractors Messrs. John Laing & Son, Ltd.

The Effect of Concentrated Loads on Concrete.

In Bulletin No. 122 of the Deutscher Ausschuss für Stahlbeton (Berlin: Wilhelm Ernst & Sohn. Price 14 D.M.), Dipl.-Ing. W. Pohle describes tests on the application of large concentrated loads to small areas such as pile-heads, reinforced concrete hinges in bridges, and anchorages for prestressing cables. The results are expressed in terms of the ratio $Z = \frac{P}{AW}$ in which P is the load at the time of failure, A is the cross-sectional area to which the load is applied, and W is the cube strength of the concrete.

The value of Z was found to vary from 3·7 to 10·1, the lower value applying to unreinforced specimens and the higher value to specimens with much helical reinforcement. In the case of concrete with low compressive strengths, the values of Z were greater than in the case of high-quality concrete. The influence of the aggregate and the volume of the pores upon the value of Z has not been established with any degree of certainty.

In the case of reinforced concrete pile-heads in which the strength of the concrete is not less than 4280 lb. per square inch, the values of Z are sufficiently high to dispense with special measures to strengthen the pile-head provided that lateral deflection is prevented.

Residential Flats at Roehampton.

The illustrations show two of a total of twenty blocks of residential flats now being built at Roehampton for the London County Council.

Fifteen of the blocks are of the type shown in *Fig.* 1. These measure 70 ft. by 47 ft. in plan and are twelve stories high. The construction is of reinforced concrete cast in place up to and including the first story. For the second and succeeding floors the columns and floor slabs are cast in place, and the beams, staircases, and landings are precast. The outer walls comprise precast slabs, with an exposed aggregate finish, of story height. Each building includes 1162 precast units. The greatest weight of a precast unit is 1 ton. The facing slabs are erected with the aid of an electric hoist on a runway cantilevered from the roof, and this is also used for pointing between the slabs and for fixing and glazing the windows.

Five of the blocks are eleven stories high (*Fig.* 2); these are 196 ft. by 40 ft. in plan, and each comprises 75 maisonettes. The construction comprises alternate frames and internal walls of story height cast in one lift in shutters measuring 33 ft. long by 8 ft. high, which are moved by a tower crane. The beams, stairs, and balconies are precast, and each building includes 1264 precast wall slabs with exposed aggregate finish.

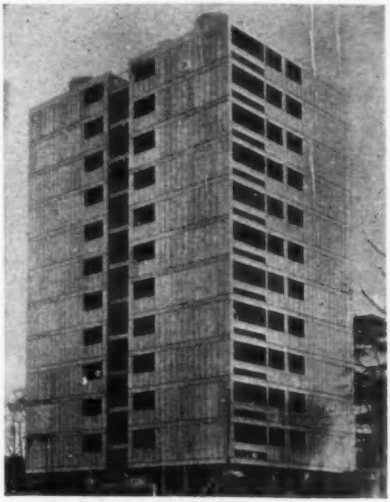


Fig. 1.

Fig. 2.

A tower crane travelling on a track is used to serve two of the structures, and the construction is arranged so that the different trades can be carried on alternately on each structure. The superstructure and external walls are being completed in a period of ten weeks.

The total value of the contract is £2,700,000. The work was designed by the Architect's Department of the London County Council, Messrs. W. V. Zinn & Associates are the consulting engineers, and Messrs. Wates, Ltd., are the contractors.

Post-Graduate Training in France.

The scarcity of engineers specialising in reinforced concrete in France has led to the formation by the Chambre Syndicale des Constructeurs en Cement Armé de France et de l'Union Française of a postgraduate course in the theory and practice of reinforced concrete. Courses will extend over a period of one academic year and will be held in Paris under the direction of a body formed for the purpose and known as the Centre de Hautes Etudes du Béton Armé et Précontraint. Further information may be obtained from the Chambre Syndicale des Constructeurs en Cement Armé, 3 rue de Lutèce, Paris, 4.

Market at Wolverhampton.

FIG. 1.

A RETAIL market is being constructed for the Borough Council on an island site near the centre of Wolverhampton. About half the site of 10,000 sq. yd. is being covered and the remainder will be surfaced for use as an open market (*Fig.* 1). Office buildings are arranged along two sides of the site. These are of beam-and-slab construction and are connected by a bridge at the upper floors. In the case of both buildings the front columns at ground level form a colonnade in front of shops. The ends of the buildings are faced with Portland stone.

Near to the office buildings is the main market which is covered by seven bays of barrel-vault roofing supported by arch ribs. Beneath the market are a parking space for 100 cars and a store. The columns supporting the market floor are on a grid of 29 ft. by 26 ft.; this spacing was determined by the planning of the car park, and the floor is designed so that the car park can be converted into an air-raid shelter.

The ventilation of the car park is provided by reinforced concrete ducts under the floor, which terminate in sheet-metal risers at each column. Provision is made for building brick air-raid shelter walls into the concrete structure by providing ¼ in. bars, at present concealed by concrete, but which can later be exposed and used for connecting the walls.

A separate market is provided for meat and fish. This is covered by three bays of north-light "shells", of 60 ft. span and 40 ft. chord. Ventilation is provided by extracting fans in the roofs and by windows in the north lights. Under these markets are kitchen and restaurant, which face on to a paved terrace and sunken garden.

The scheme was designed by the Borough Architect's Department; Messrs. William Moss & Sons, Ltd., Loughborough, are the main contractors. The reinforced concrete design was prepared and high-tensile reinforcement is being supplied by G. K. N. Reinforcements, Ltd., who also carried out the site investigation.

A New Advisory Service.

THE Universal Asbestos Manufacturing Group, of Tolpits, Watford, has started a free advisory service on the design and use of building materials made of asbestos cement, pitch fibre, and reinforced plastics.